המועצה לשימור
אתרי מורשת בישראל

Local and global models for seismic safety assessment

Paulo B. Lourenço

pbl@civil.uminho.pt
www.civil.uminho.pt/masonry

Universidade do Minho

Introduction

isise

Universidade do Minho

Introduction

- Modern societies believe that cultural heritage buildings are landmarks of culture and diversity. They should last forever. This act of culture poses high demands to all because deterioration is intrinsic to life.



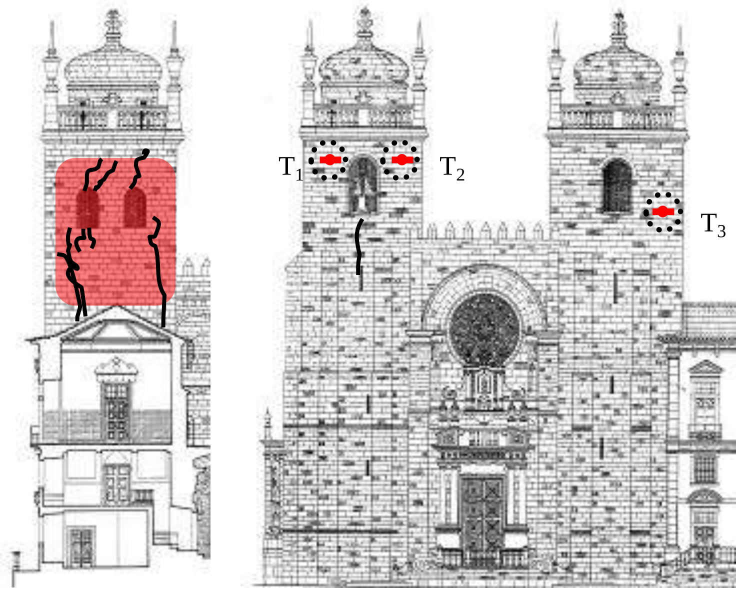


Introduction: Buildings that must live forever (I)

Venice, 1902

Noto, 1996

Introduction: Buildings that must live forever (II)



Cathedral of Porto

Bam Earthquake

Introduction: Buildings that must live forever (IV)

- 1755: One of the largest earthquakes in the world strikes Lisbon (estimated magnitude 9.0)
- Buildings lost but also movable heritage:

  At the Trindade convent, various precious cult artefacts, organs and a large library

  At the S. Domingos convent, several precious furniture, silver and gold artefacts, and several libraries with 15000 volumes in golden binding

  At the S. Francisco monastery, all silver was melt and a library of 9000 volumes lost

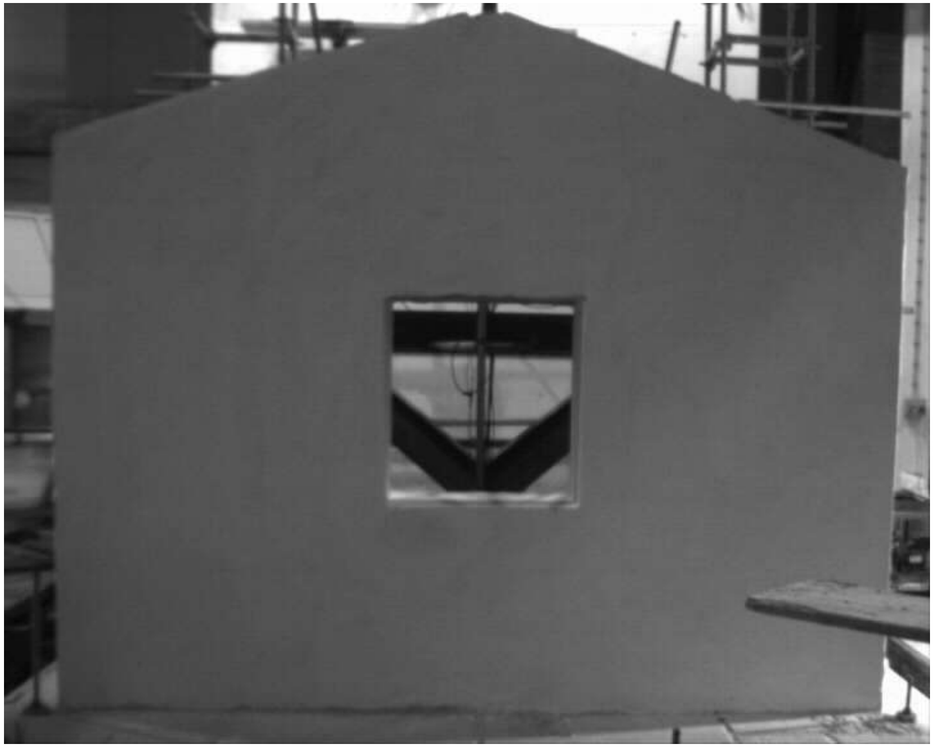
  At the Espírito Santo monastery, a precious diamond custody and the large library burnt.

Earthquake, fire and tsunami

Carmo church ruins

Recent Tests: Flexible Diaphragm (Non-Strengthened)

- Rubble masonry wall
- Full collapse



The Role of the Engineer

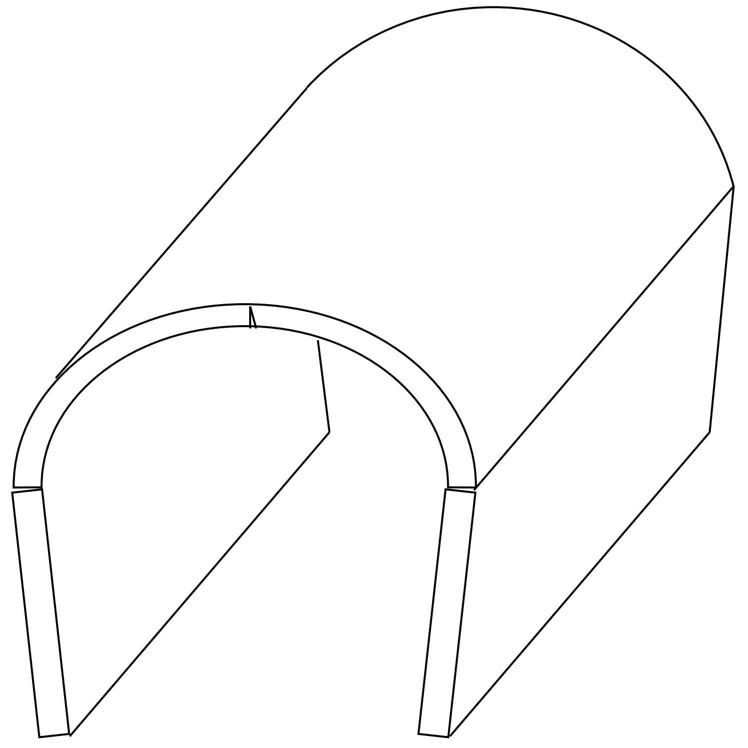
- **Rehabilitation and conservation of the built environment takes about 35% of the construction market in Europe, reaching 50% in several countries**
- **“Conservation engineering” is difficult and requires a different approach and skills from those employed in designing new construction:**
  - **Complexity (scatter of properties, lack of original design elements / Non-conforming execution, deficient structural connections, load transfer…)**
  - **Different knowledge (materials, technologies, …)**
  - **Lack of education in regular engineering / architecture courses**
  - **Non-applicable codes**
  - **Advanced structural analysis tools have justification**

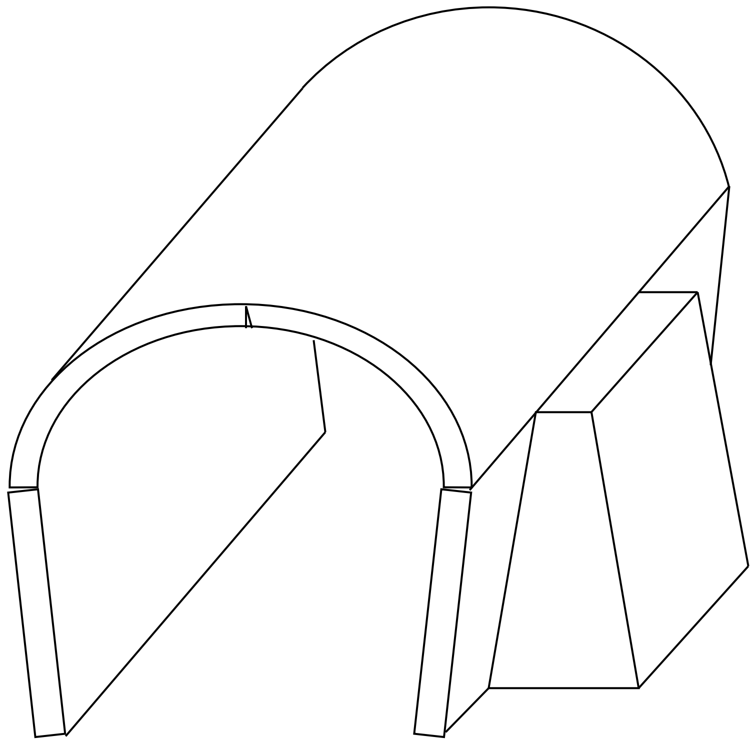
Why “Conservation Engineering”?

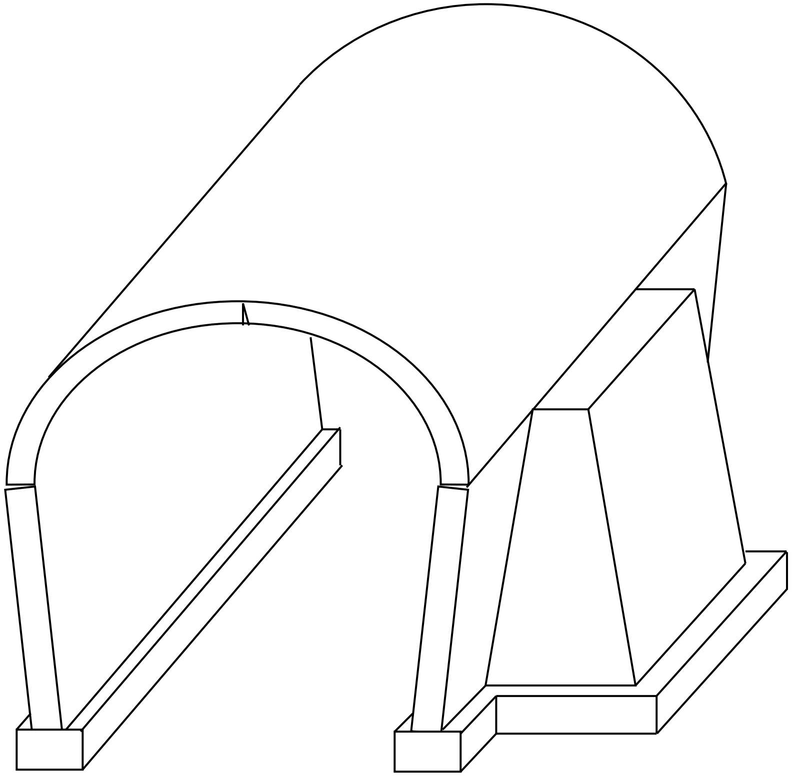
- Those involved in historic preservation must recognize the contribution of the engineer. Often engineering advice seems to be regarded as something to be sought at the end of a project when all the decisions have been made, while it is clear that better solutions might have been available with an earlier engineering contribution.
- Conservation engineering requires a different approach and different skills from those employed in designing new construction. Often historic fabric has been mutilated or destroyed by engineers who do not recognize this fact, with the approval of the authorities and other experts involved. Moreover, even when conservation skills are employed, there are frequent attempts by regulating authorities and engineers to make historic structures conform to modern design codes. This is generally unacceptable because the codes were written with quite different forms of construction in mind, because it is unnecessary and because it can be very destructive of historic fabric.

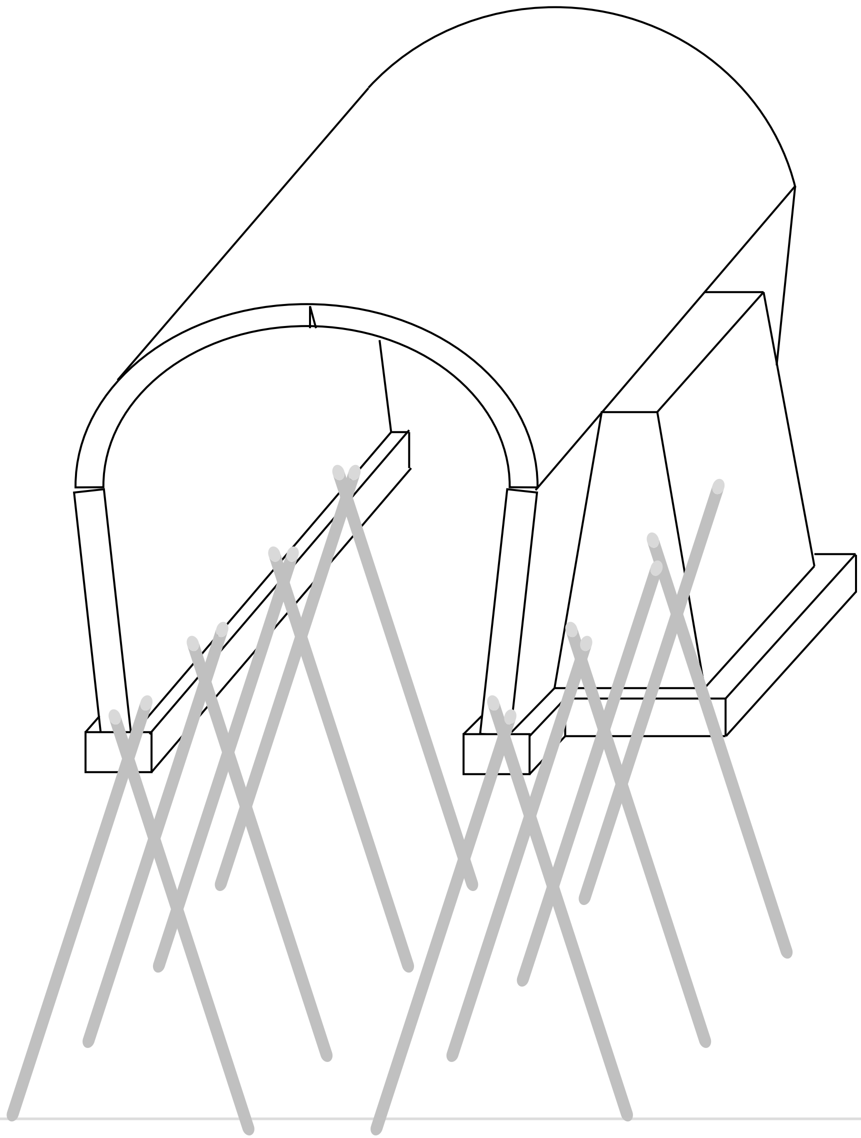
Why “Conservation Engineering”?

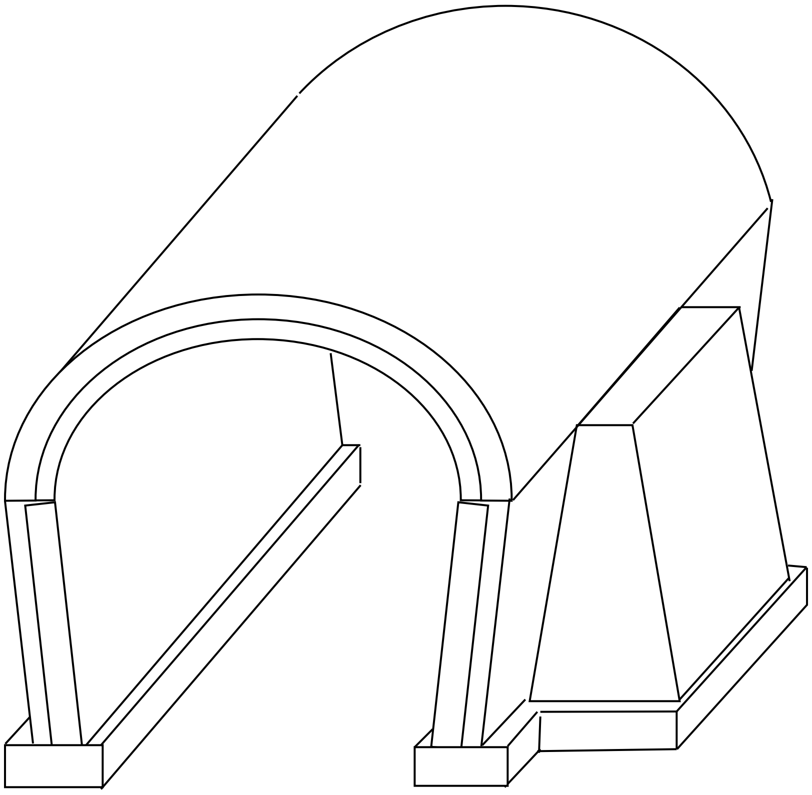
- The need to recognize the distinction between modern design and conservation is also of relevance in the context of engineers’ fees. The usual fee calculation based on a percentage of the cost of the work specified is clearly inimical to best conservation practice, with the ideal is to avoid any structural intervention if possible. Being able to recommend taking no action might actually involve more investigative work and hence more cost to the engineer than recommending some major intervention.
- Modern intervention procedures require a thorough survey of the structure and an understanding of its history. Any heritage structure is the result of the original design and construction, any deliberate changes that have been made and the ravages of time and chance. An engineer working on historical buildings must be aware that much of the effort in understanding their present state requires an attempt to understand the historical process. The engineer involved at the beginning of the process might not only have questions that can easily be answered by the archaeologist or architectural historian, but he might be also able to offer explanations for the data being uncovered.

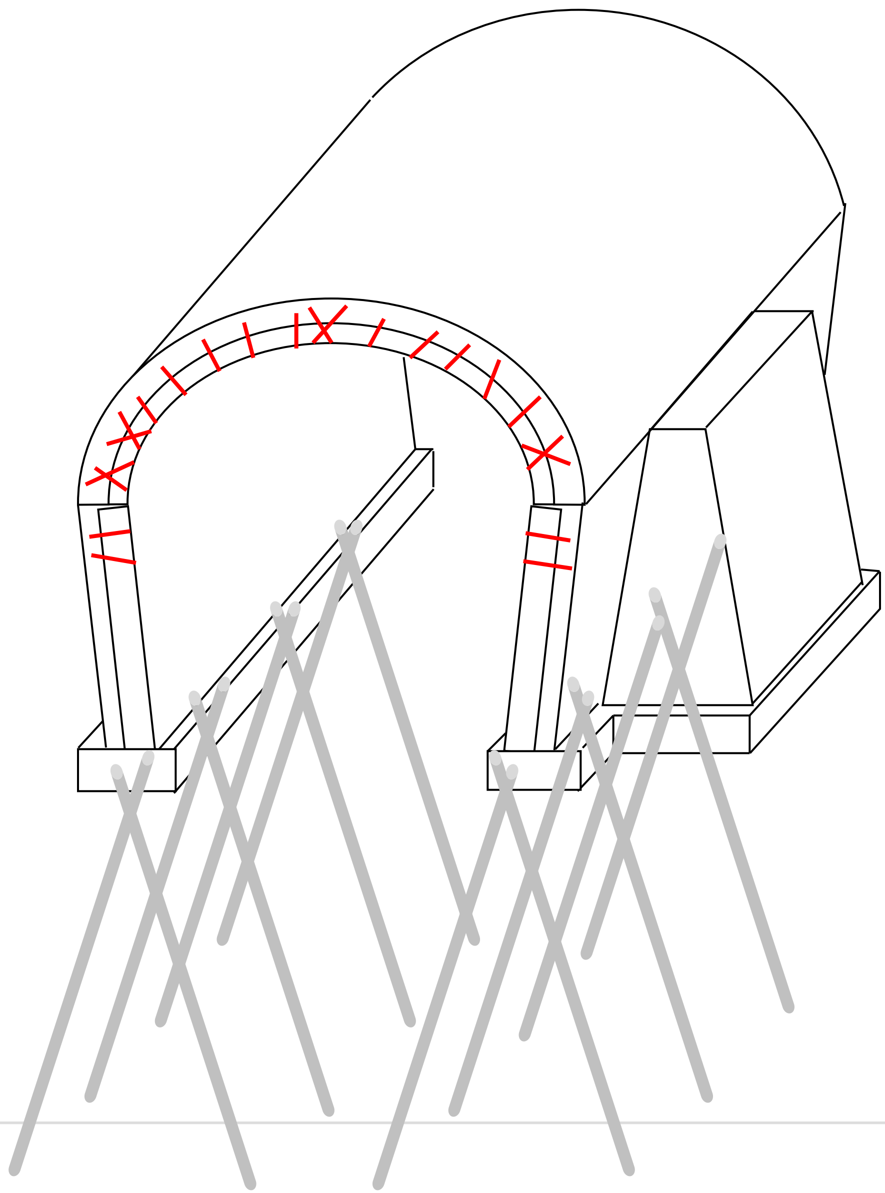


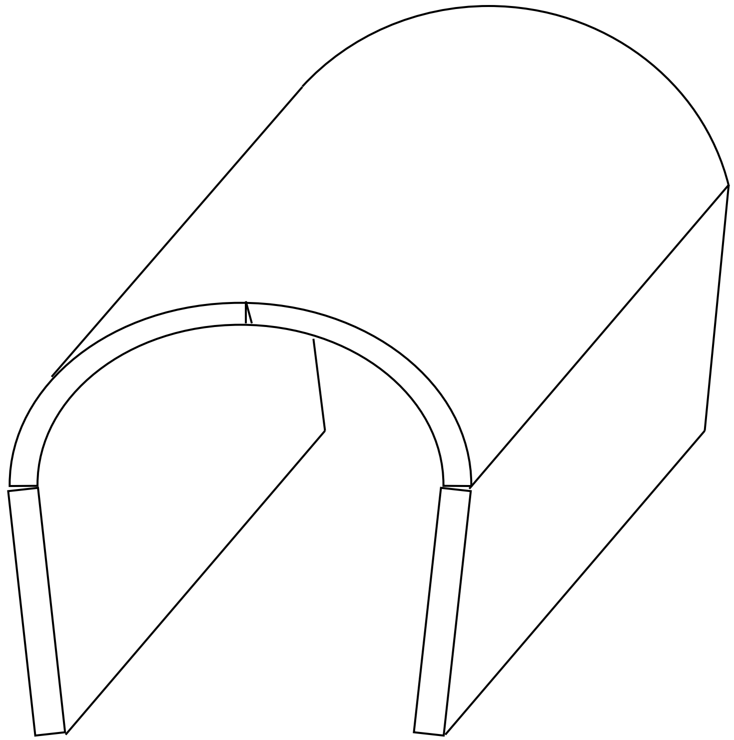


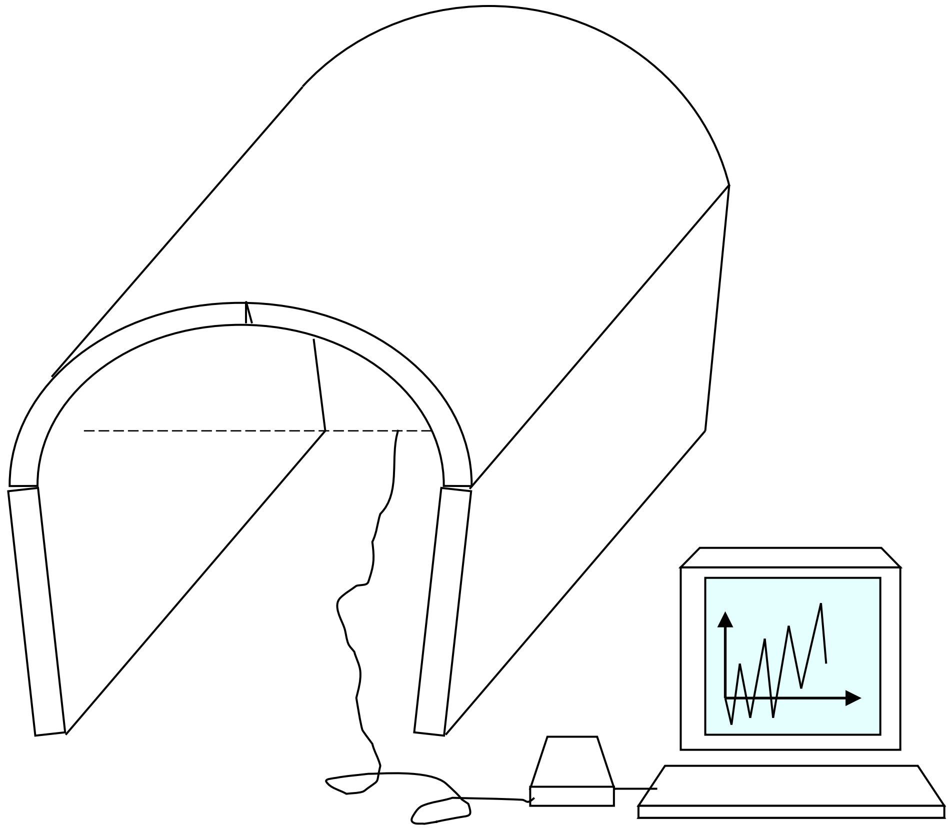


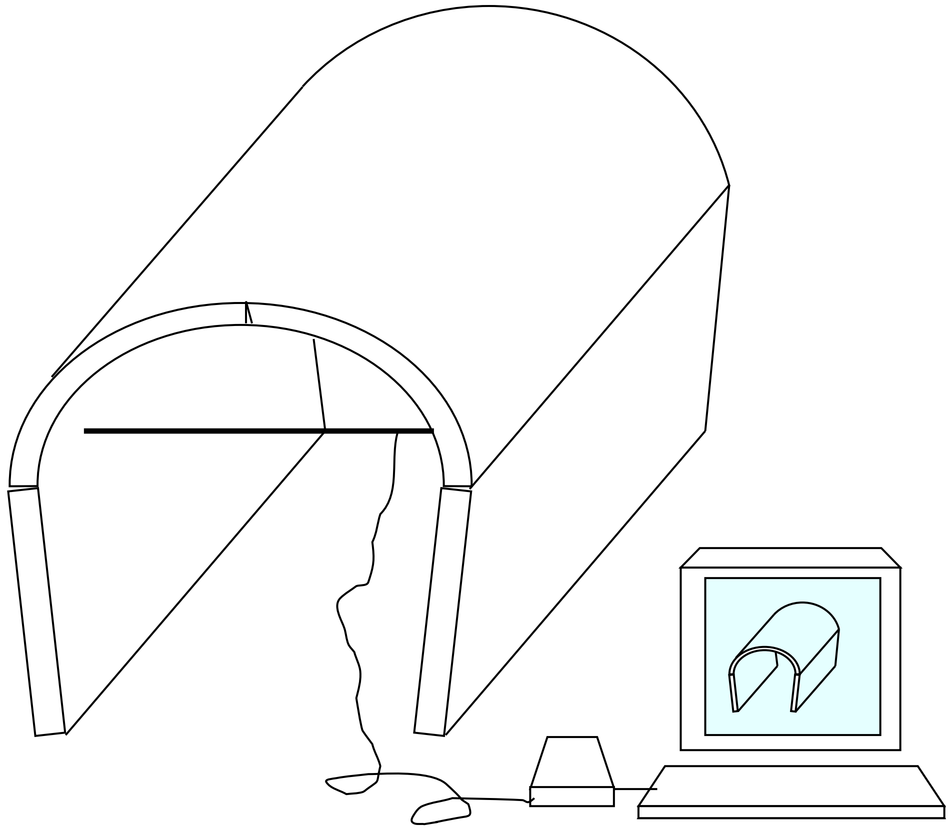


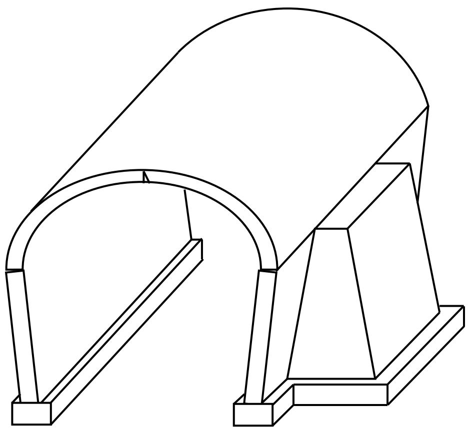












Past understanding

"Conservation" is warranted by the powerfulness of the intervention

Blind confidence in modern materials and technologies

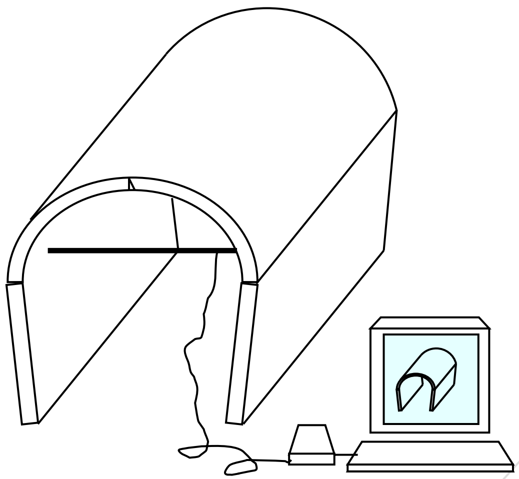
Mistrust towards original or ancient materials and original resisting resources of the building

The value of original / ancient structure and structural principles is not recognized

The importance of previous studies is not fully recognize

Significant negative experience accumulated

Athens Charter (1931)
Recommends the use of concrete and other modern material and techniques for restoration purposes. Added materials and components should be hidden to avoid altering the historical aspect of the building.



Modern understanding

Respect towards authenticity of the structure and structural principles governing its response

Conservation should lye on knowledge and understanding of the nature of the structure and real causes of possible damage or alterations

Minimal and respectful interventions (minimal, non-intrusive and reversible)

Importance of previous study (comprising historical, material and structural aspects)

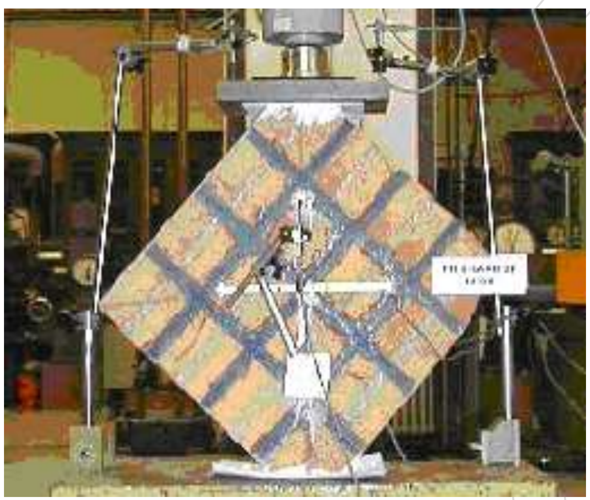
The previous study and the intervention are multidisciplinary tasks requiring the cooperation of historians, architects, engineers, physicists,…

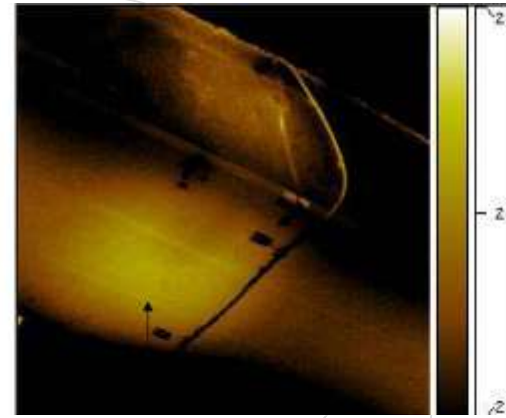
Venice Charter (1964)
Recommends the use of traditional or historical materials for stabilization or restoration. Suggests the use of modern materials / techniques for cases where it is not possible to stabilize or restore by means of traditional / historical techniques.

It must be possible to distinguish new materials or components from the original ones.

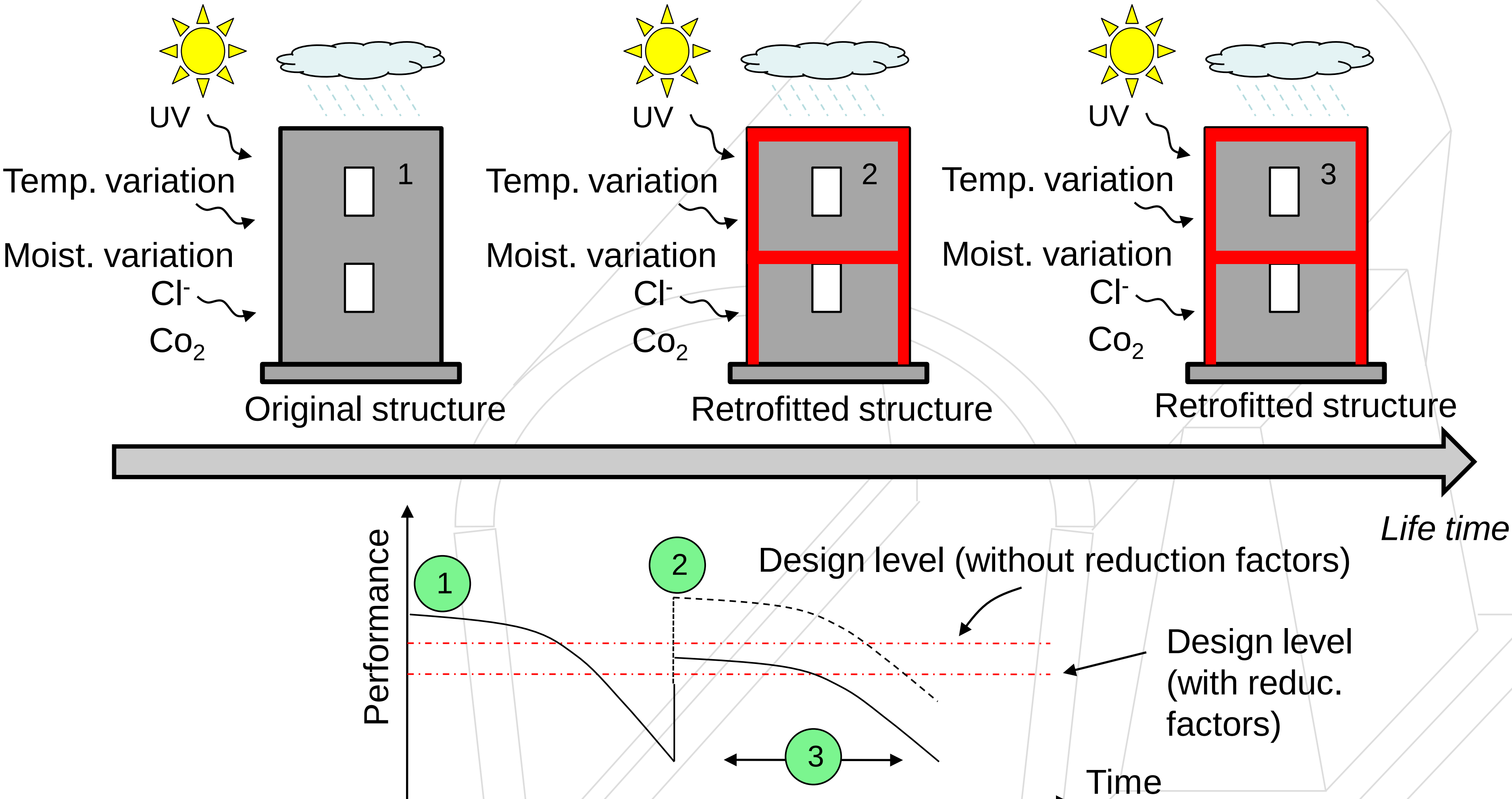
Unfavorable properties of cement mortars:

1. Brittleness and high strength
2. Difficult to remove if needed
3. Thermal expansion coefficient can be twice that of lime mortars and brick / stone
4. Low porosity and especially the large amount of small pores (hinder water movement in masonry and cause damage due to the accumulation of moisture behind cement layers or to evaporation and deposition of salts in adjacent stones or bricks)
5. Soluble salts such as calcium sulphates and sodium salts are often present in cement mortar, and leach out over time. Lime mortar has a low efflorescence potential due to high chemical purity
6. Lime mortar allows limited movement within the joints and can undergo autogeneous healing due to dissolution and precipitation processes
7. Lime mortar is softer and more porous than masonry, acting as a sacrificial substrate where evaporation of water and associated decay from soluble salt crystallization can take place




27
26
25

Environmental degradation



THE PHASES OF THE STUDY

Diagnosis — Identify causes of damage and decay

↓

Safety evaluation — Determine acceptability of safety levels by analyzing present condition of structure and materials

↓

Design of remedial measures — Lay-out repair or strengthening actions to ascertain the required safety

Diagnosis and safety evaluation of the structure are two consecutive and related stages on the basis of which the effective need for and extent of treatment measures are determined. If these stages are performed incorrectly, the resulting decisions will be arbitrary: poor judgement may result in either conservative and therefore heavy-handed conservation measures or inadequate safety levels.

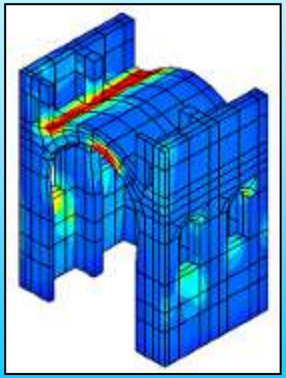
Methodology

HISTORY

INSPECTION
PRESENT CONDITION

MONITORING

STRUCTURAL ANALYSIS



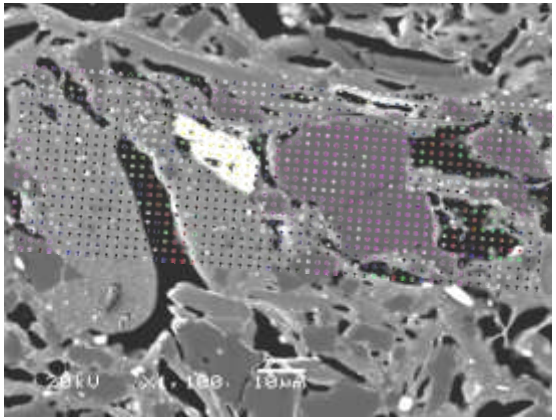
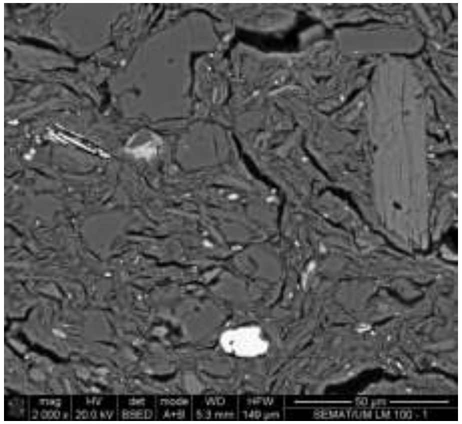
Model validation

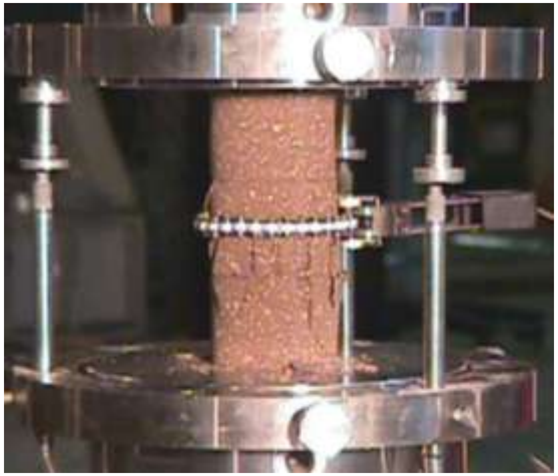
EVIDENCE

HYPOTHESES

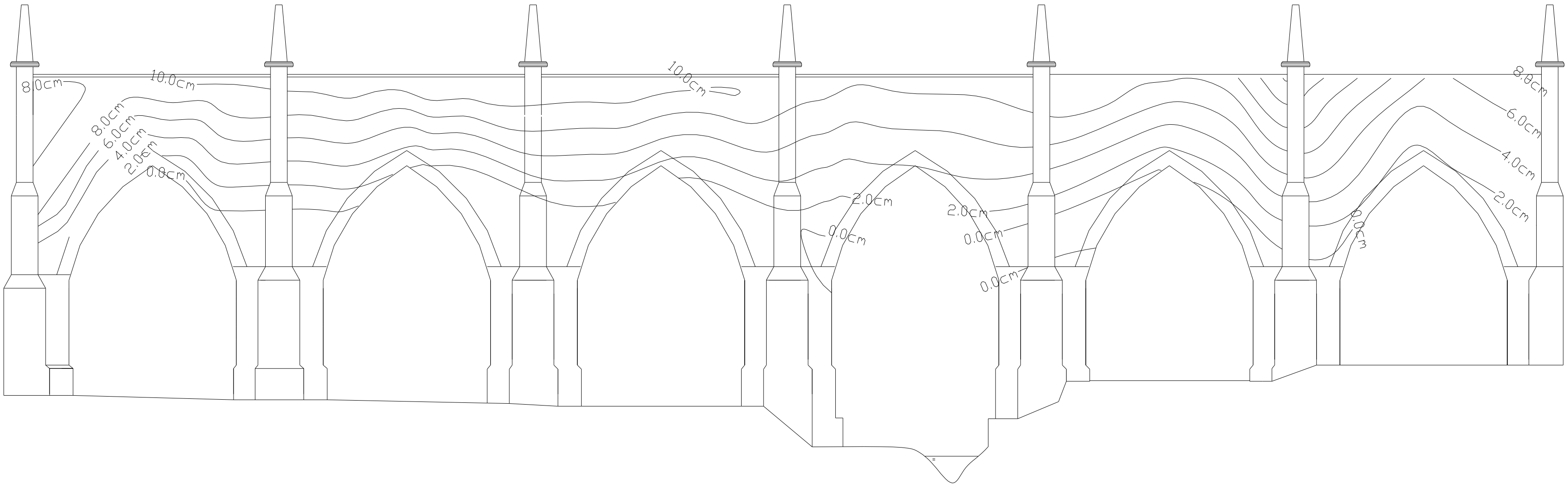
Conclusions on building condition and adequate remedial measures

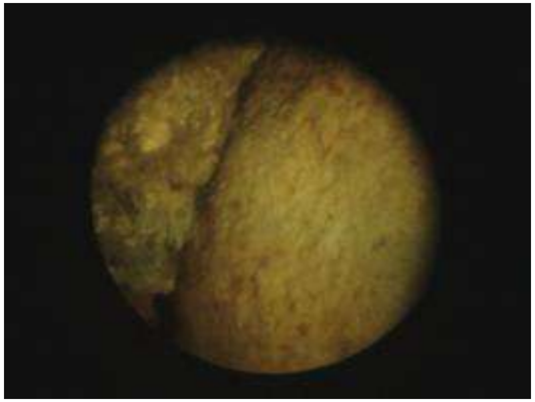
The need of experimental knowledge

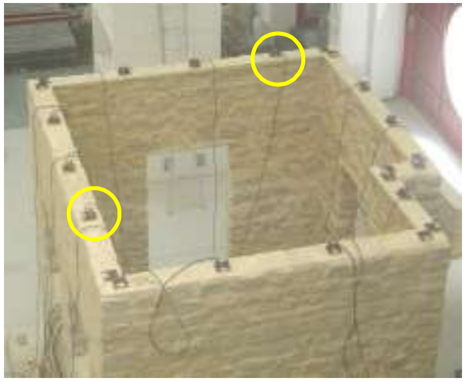




Survey and visual inspection



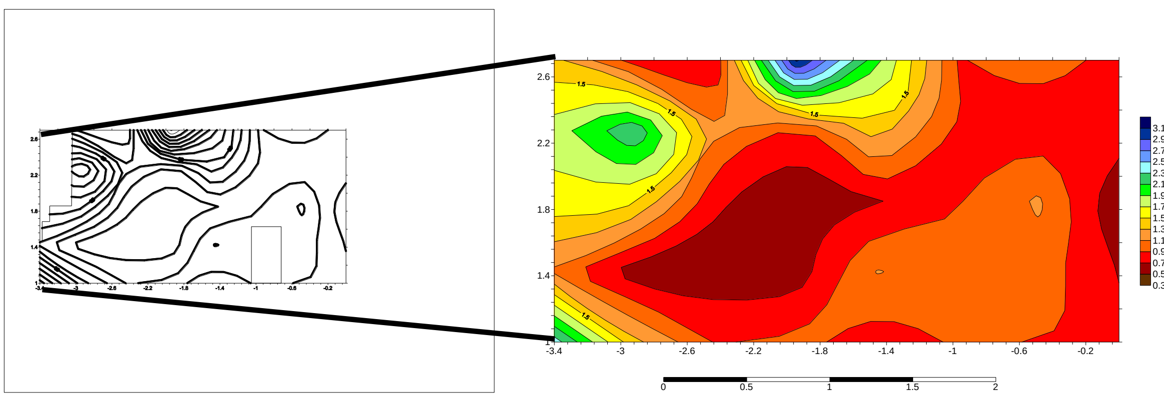


NDT & Identification

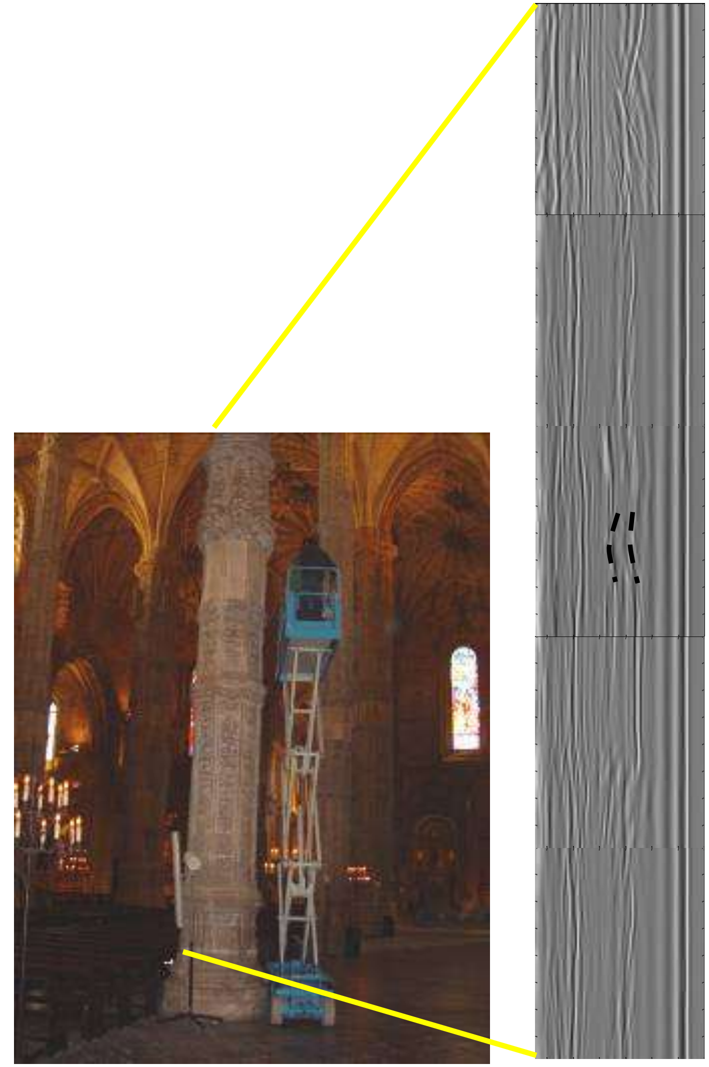


Measurements

Strain gauge



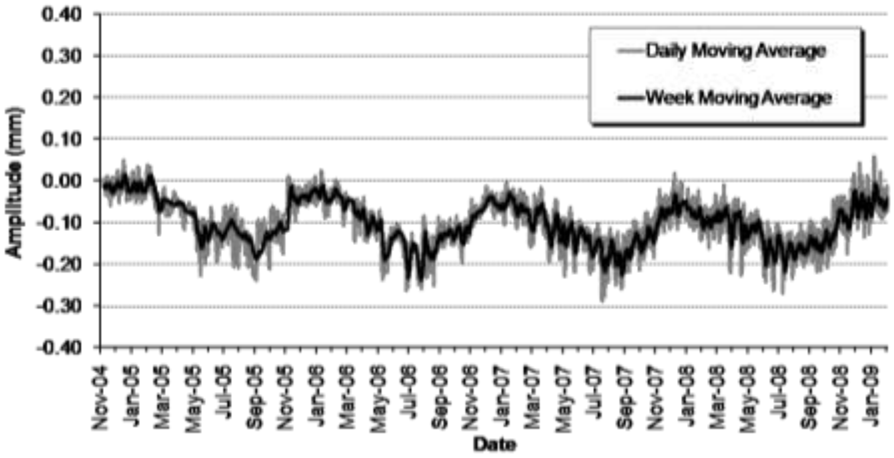
Sonic tomography

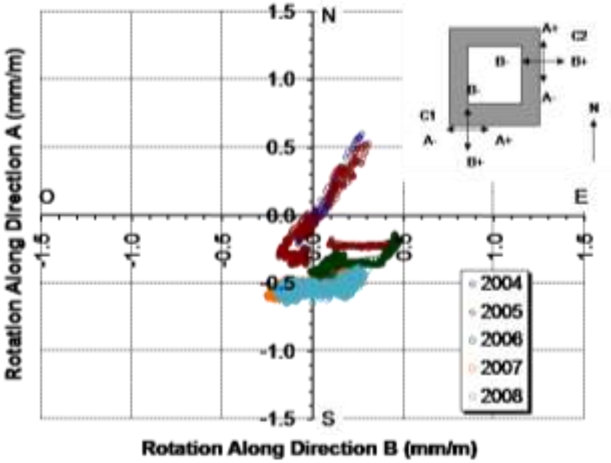


GPR testing

Monitoring

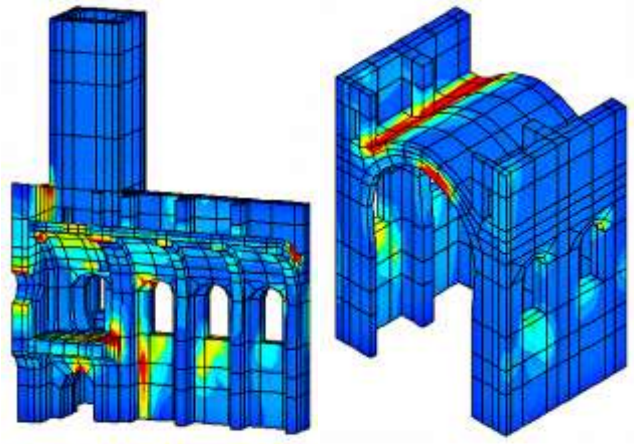


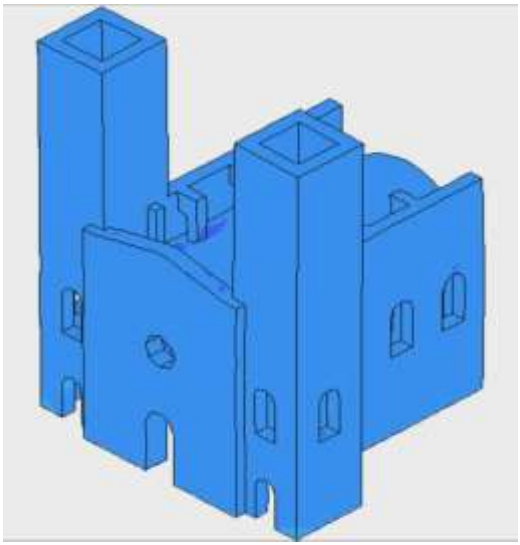


Crack opening and tilting, Cathedral of Porto

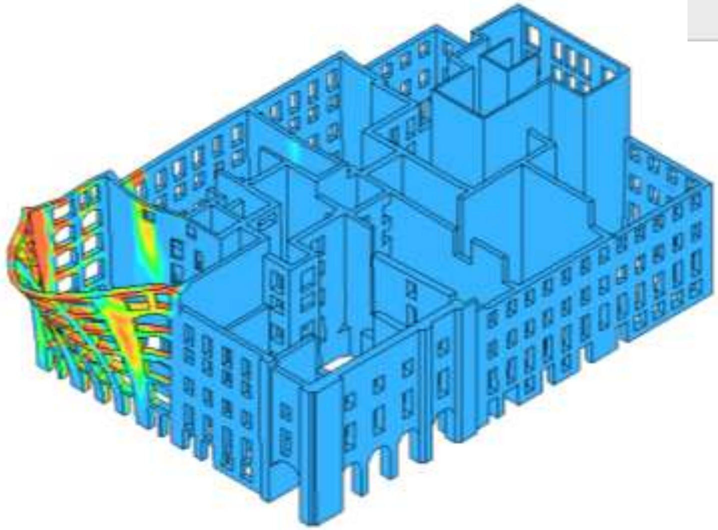
Solving Engineering Problems & Definition of Practical Rules



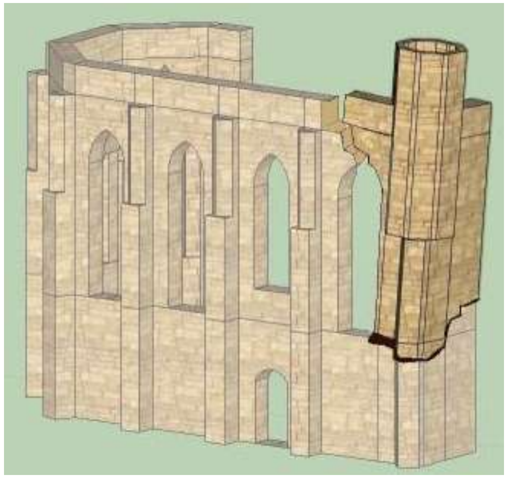
Settlements



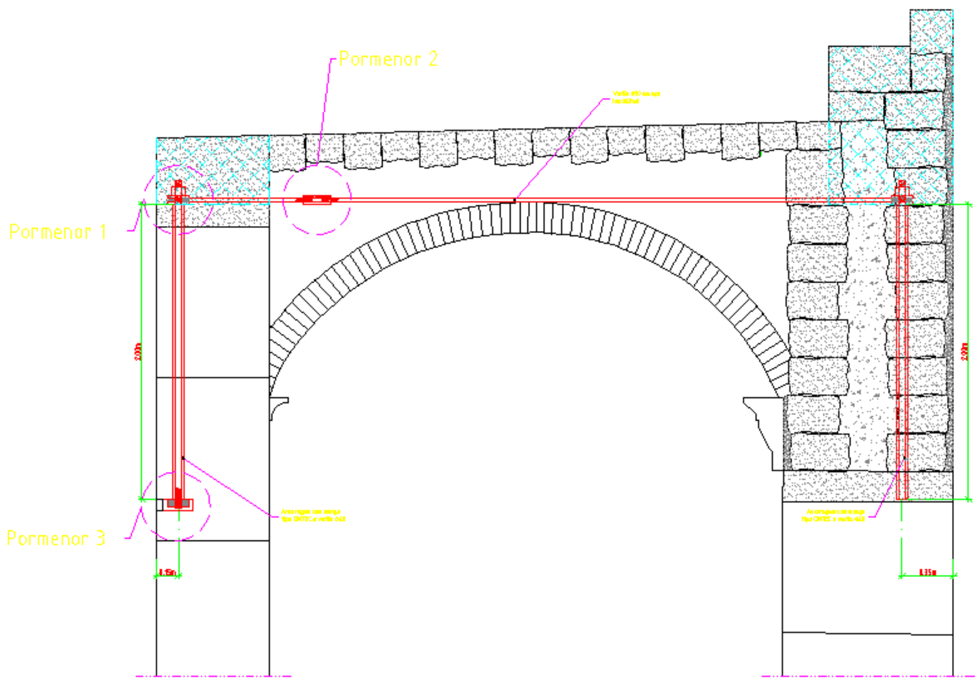
Vehicles

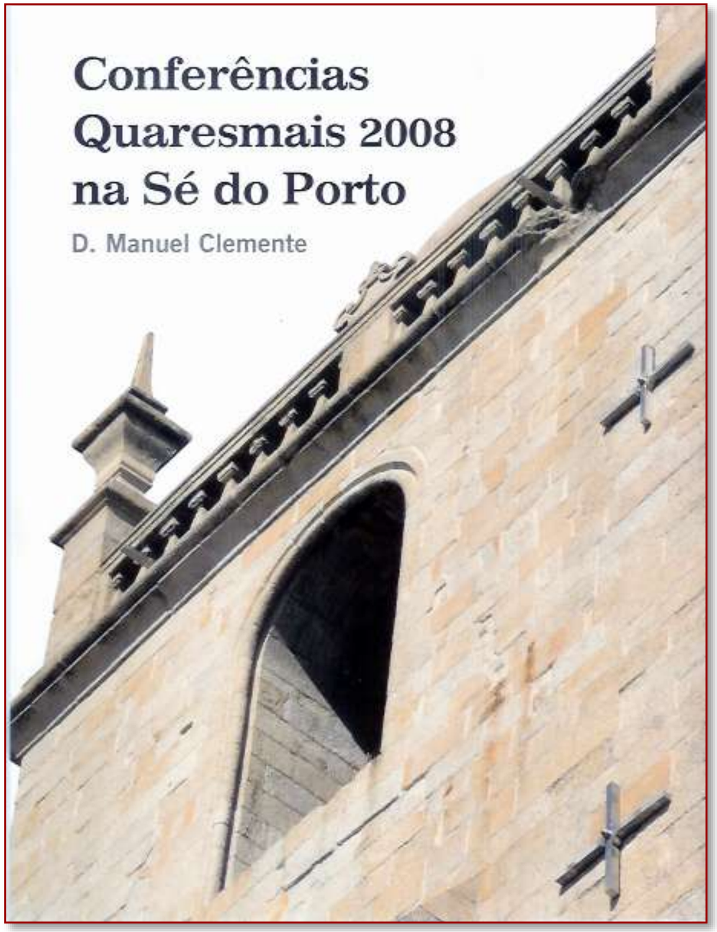


Earthquakes

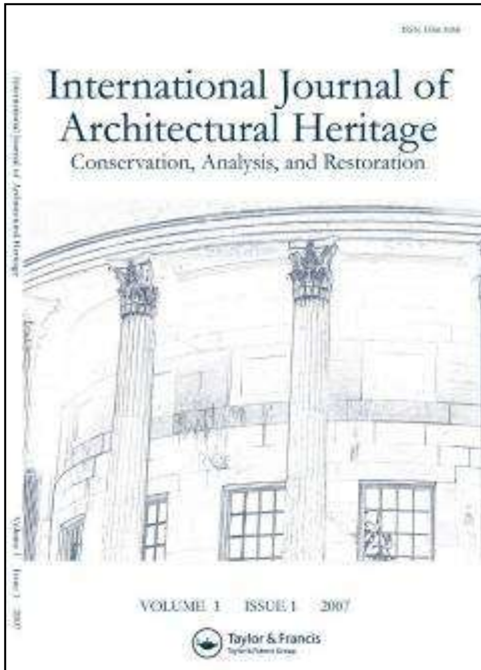


Remedial Measures



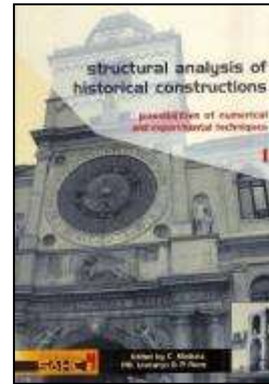


Education (I)



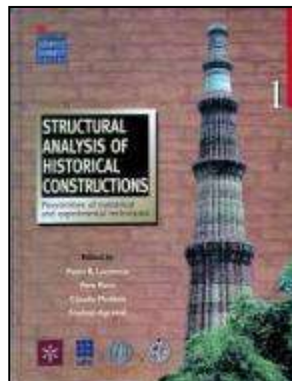
Taylor and Francis,
since 2007
(4 numbers/year)

Guimarães, 2001
500 participants

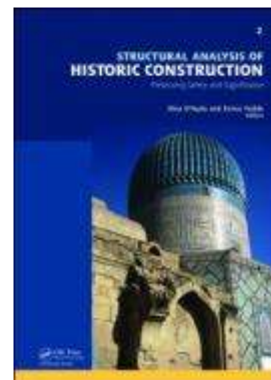


Padua, 2004
350 participants

Shanghai, 2010
250 participants



New Delhi, 2006
300 participants



Bath, 2008
250 participants

Wroclaw, 2012
350 participants

Conference Series: Structural Analysis of Historical Constructions

Education (II)

- MSc Course: Structural Analysis of Monuments and Historical Constructions: 150 students from 50 countries




Six edition in 2012/2013
secretariat@msc-sahc.org
www.msc-sahc.org

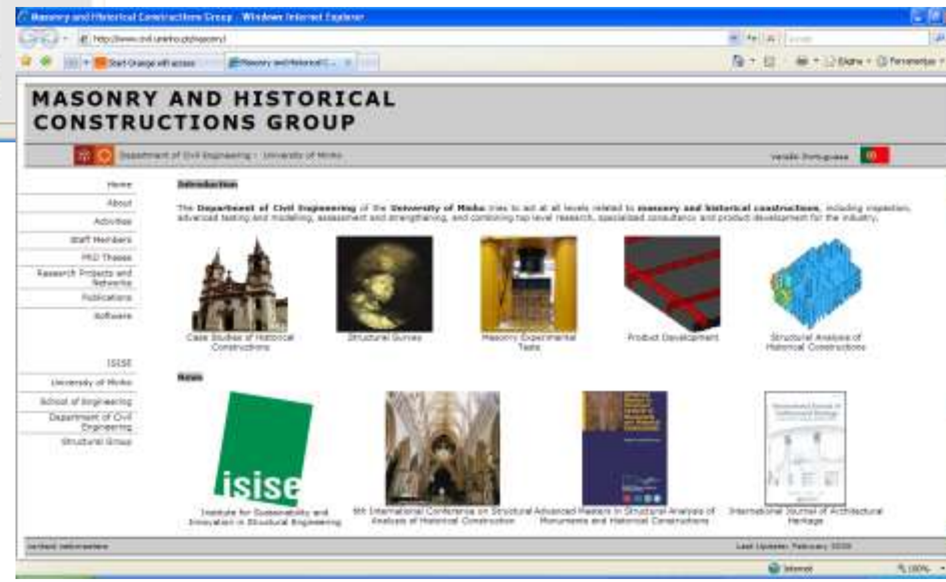
Grants for students:
Between 16.000 and 24.000 euro/year.
Available for Israeli students

Grants for scholars:
1.200 euro/week

Education (III): Webpage

www.isise.net

www.civil.uminho.pt/masonry



המועצה לשימור
אתרי מורשת בישראל

The Role of Structural Analysis

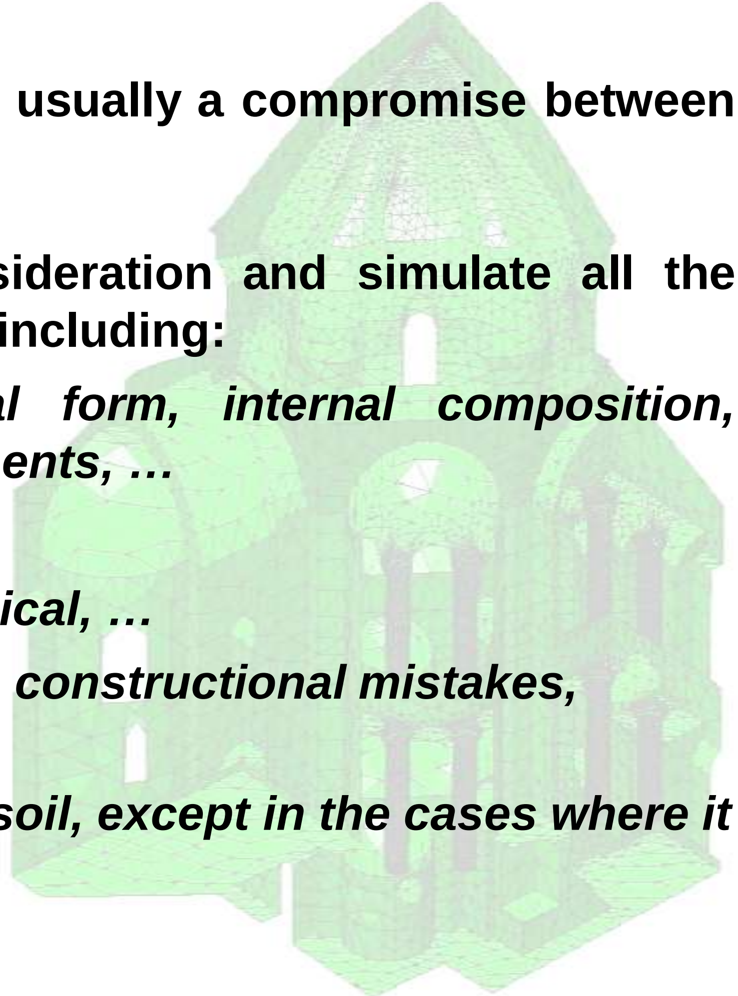
isise

Universidade do Minho

Purpose and possibilities of numerical analysis

- Role of structural analysis in the general study of an heritage structure
  - Contribution to diagnosis
  - Relationship with history, inspection and monitoring
  - Contribution to safety evaluation
  - Contribution to design / validation of intervention
- Modeling
  - Nature and type of models
  - Construction of the model
  - Need for validation
  - Uncertainties linked to prognosis
- Challenges posed by historical / heritage structures
  - Geometry, materials, actions and history

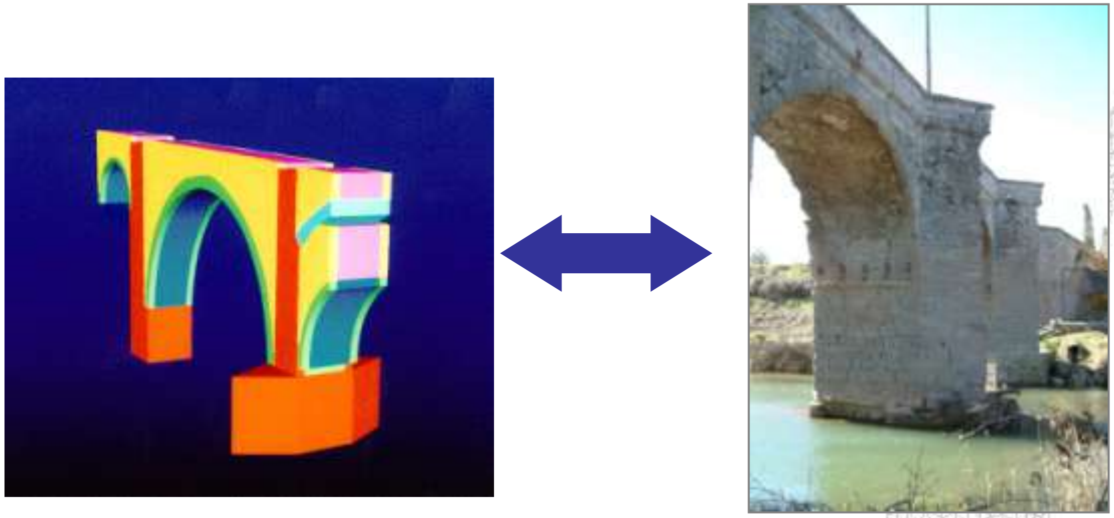
The role of structural analysis



The model used in the structural analysis is usually a compromise between realism and cost

The structural model must take into consideration and simulate all the aspects influencing the structural response, including:

- ***Geometry and morphology: structural form, internal composition, connections between the structural elements, …***
- ***The material properties***
- ***The actions: mechanical, physical, chemical, …***
- ***Existing alterations and damage: cracks, constructional mistakes, disconnections, crushing, leanings, …***
- ***The interaction of the structure with the soil, except in the cases where it is judged to be irrelevant***



No model does represent the full reality

We need models to reduce reality to a limited number of hypothesis or concepts (and to work with them)

We need models to predict responses from our concepts or hypothesis

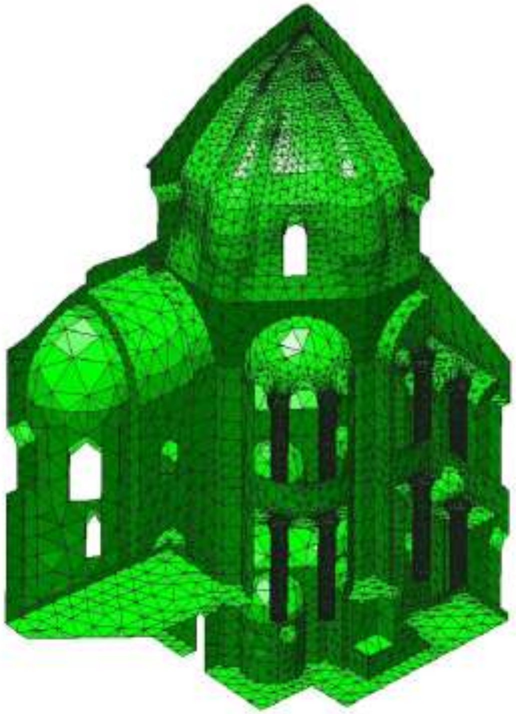
Models must be validated

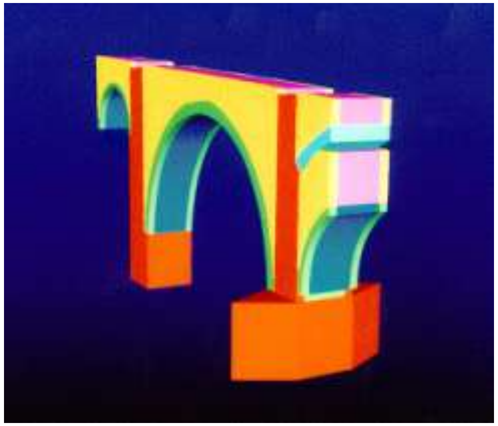
The possibilities of models are always limited, but models are our best guess

THE STRUCTURAL MODEL IS THE RECIPIENT OF OUR HYPOTHESIS

(1) the fundamentals of the description of the mechanical / strength response

(2) specific quantities related to material, geometrical, morphological properties





STRUCTURAL MODEL

1ST PHASE

PREDICTIONS

ALLOWING COMPARISON & CALIBRATION

2ND PHASE

PREDICTIONS USED IN EVALUATION

Safety evaluation

CHALLENGE:

Trying to comply with the principle of minimum intervention, while maintaining an acceptable level of risk

SAFETY EVALUATION: DIFFICULTIES

1- LIMITED APPLICABILITY OF AVAILABLE CODES

Codes prepared for the design of modern structures are often inappropriately applied to historic structures. They are based in calculation approaches which may fail to recognize the real structural behaviour and safety condition of ancient constructions

The enforcement of seismic and geotechnical codes, can lead to drastic and often unnecessary measures that fail to take into account the real structural behaviour

2- SUBJECTIVITY AND UNCERTAINTY

Any assessment of safety is affected by two types of uncertainties

The uncertainty attached to data (actions, geometry, deformations, material properties…), used in the research.

The difficulty of representing real phenomena in a precise way with an adequate mathematical model (models provide only a limited representation of reality).

The subjective aspects involved in the study and evaluation of a historic building may lead to conclusions of uncertain reliability

LEGAL ISSUES

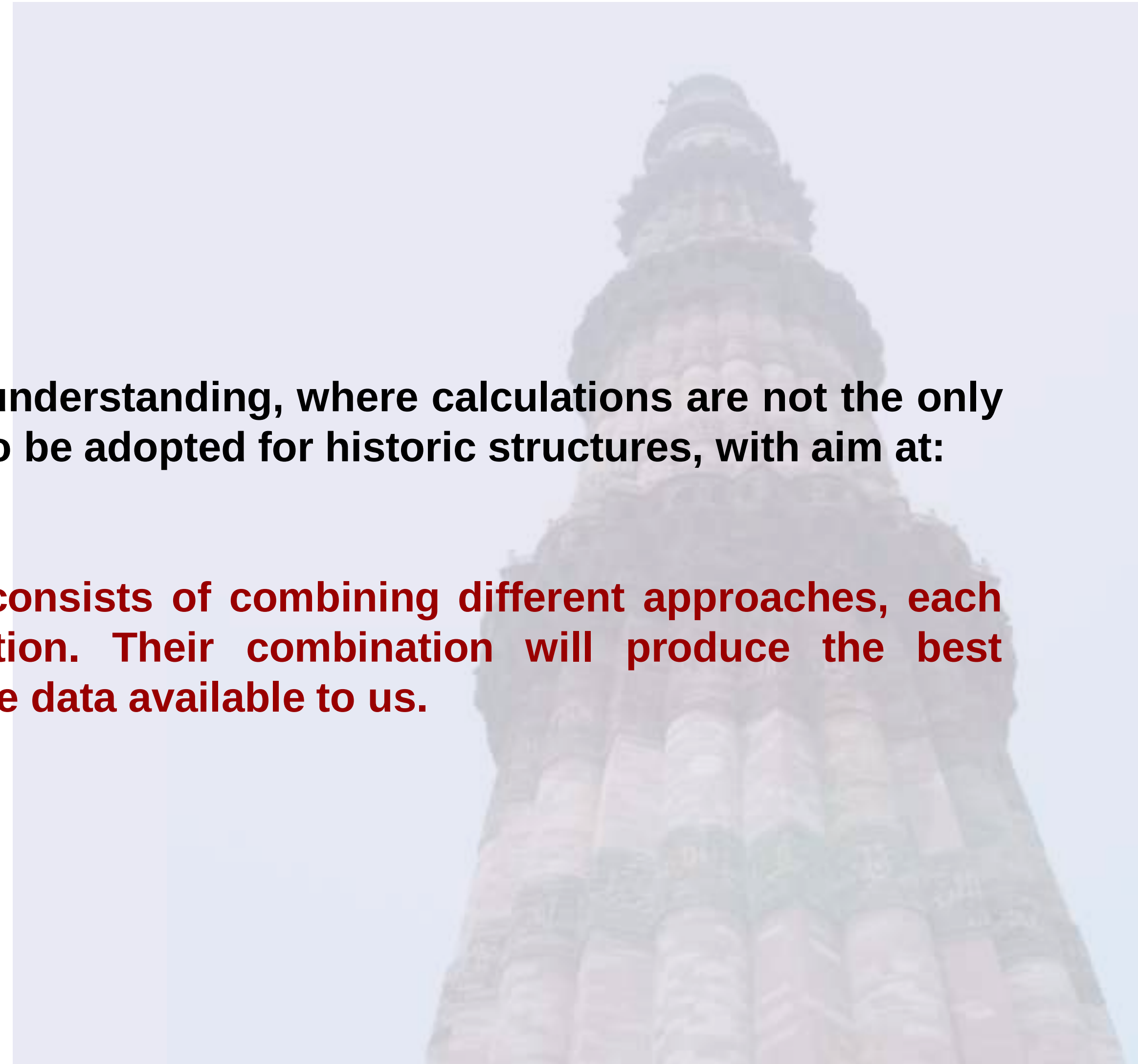
Modern legal codes and professional codes of practice adopt a conservative approach involving the application of safety factors to take into account the various uncertainties. This is appropriate for new structures where safety can be increased with modest increases in member size and cost.

However, such an approach is not appropriate in historic structures where requirements to improve the strength may lead to the loss of historic fabric or to changes in the original conception of the structure.

A more flexible and broader approach, where calculations are not the only source of evaluation, needs to be adopted for historic structures to relate the remedial measures more clearly to the actual structural behaviour and to retain the principle of minimum intervention, avoiding in any case risks for the human life.

It must be clear, therefore, that the architect or engineer charged with the safety evaluation of an historic building should not be legally obliged to base his decisions solely on the results of calculations because, as already noted, they can be unreliable and inappropriate.

SAFETY EVALUATION
PROPOSED APPROACH



A more flexible and broader understanding, where calculations are not the only source of evaluation, needs to be adopted for historic structures, with aim at:

The broader understanding consists of combining different approaches, each giving a separate contribution. Their combination will produce the best possible ‘verdict’ based on the data available to us.

SAFETY EVALUATION: POSSIBLE APPROACHES

HISTORICAL APPROACH

Knowing from history (True–scale experiment)

QUALITATIVE APPROACH

Inductive procedure (Comparing and extrapolating from other buildings)

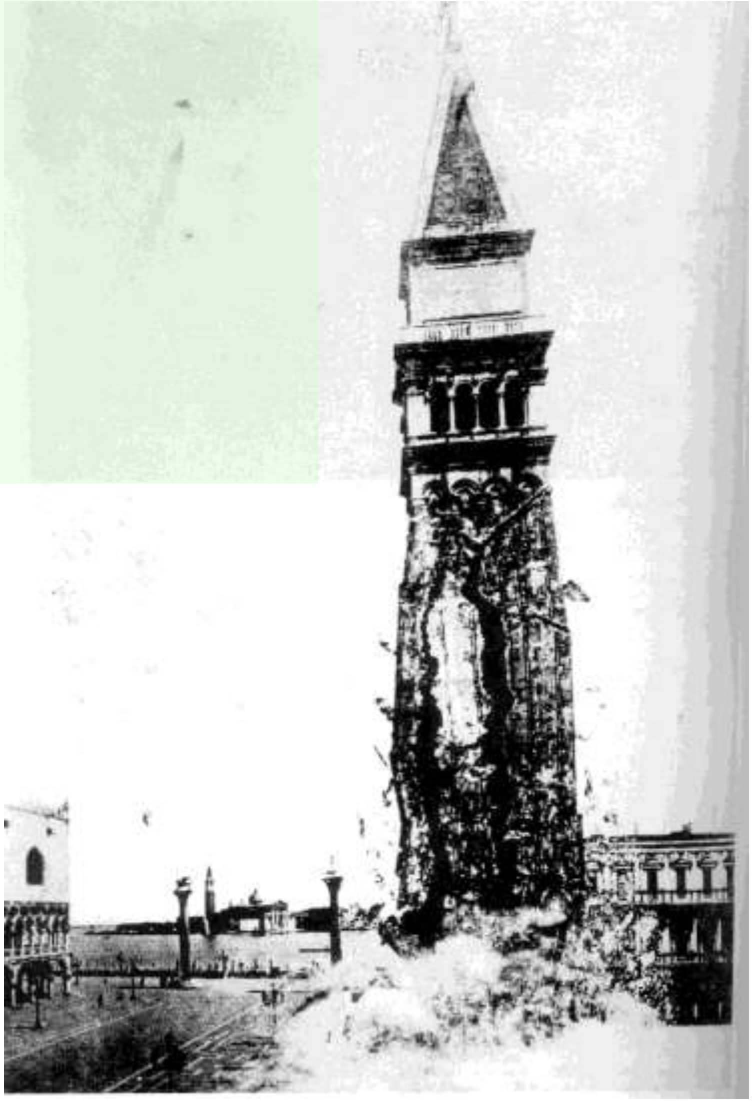
ANALYTICAL APPROACH

Deductive procedure (Structural analysis)

EXPERIMENTAL APPROACH

(Experiments on individual components or the entire building)

Historical approach



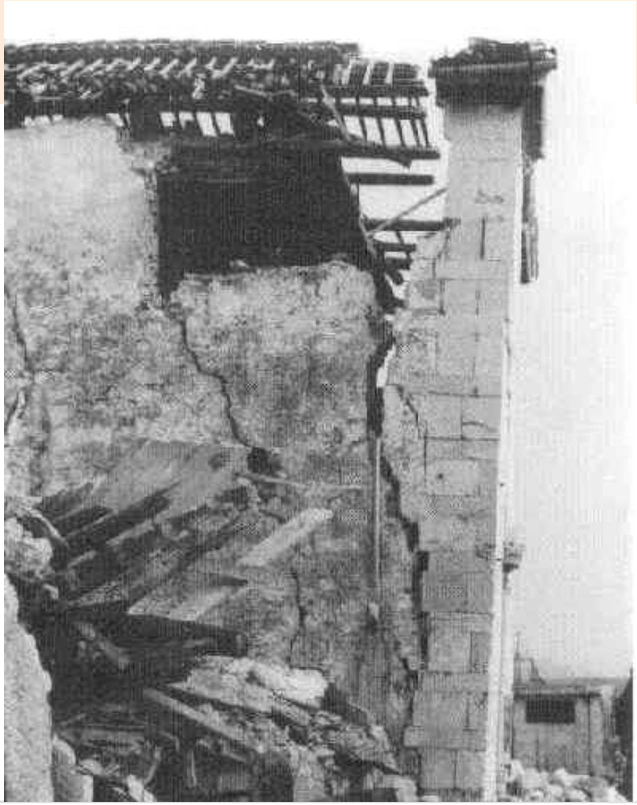
Knowing from history

Full-scale / Real time experiment

Knowing from the behaviour shown by the same structure, or similar ones, in the occasion of historical actions

Qualitative approach

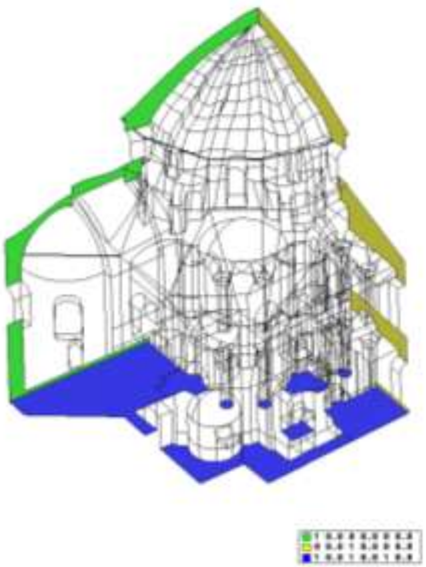
Inductive procedure (Comparing and extrapolating from other buildings)

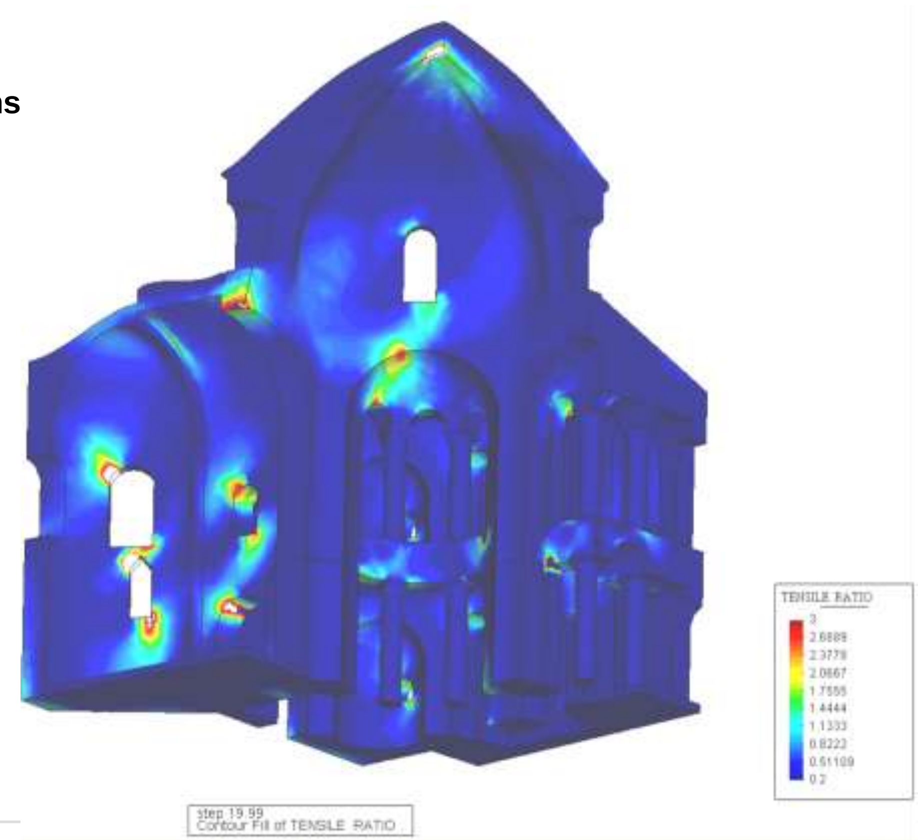


Analytical approach

Deductive procedure (Structural analysis)

Modelling & analyzing a structure to obtain quantitative predictions on the response subjected to different actions





Experimental approach

Experiments on the entire building or individual components

Example: load tests in roof-slabs or vaults

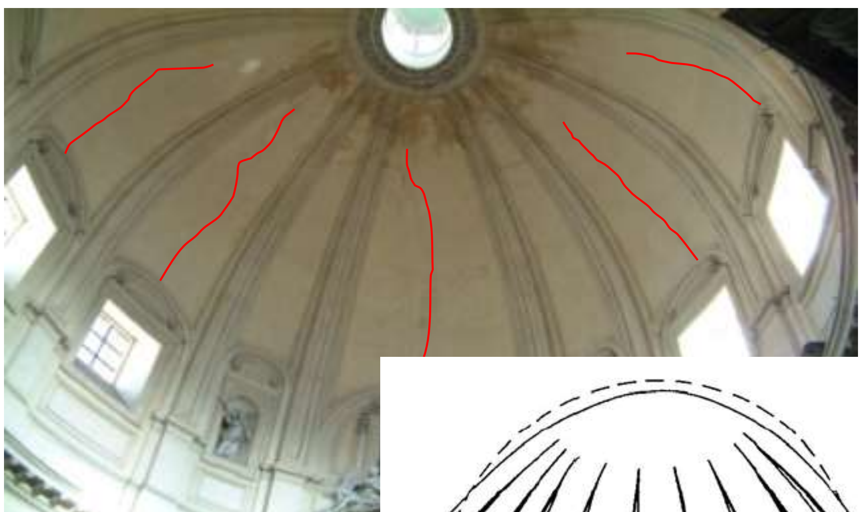
In short…

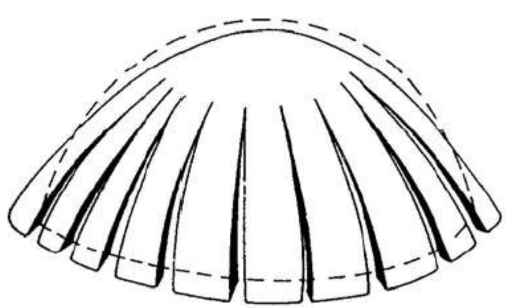
- Scientific approach
- Combination of different sources and approaches
- Methodological consistency

  Using similar approaches for diagnosis, safety evaluation and design of intervention
- Subjectivity is still possible
- Importance of personal judgment

  Recognize the need for experts and the value of their personal judgment

What not to do (I)?





The need to understand materials, structural arrangements and construction techniques from existing buildings

What not to do (II)?

It is necessary to adopt adequate safety evaluation procedures (history, quantitative analysis, qualitative analysis, experimental analysis)

What is a masonry structural system?

המוֹעָצָה לְשִׁימוּר
אתרי מורשת בישראל

isise

Universidade do Minho

Masonry Materials – Properties of Units and Mortar

- Strong variability

- Typical ancient masonry stones
  - Igneous – Granite (40 to 150 N/mm^2)
  - Sedimentary – Limestone (10 to 100 N/mm^2)
  - Metamorphic – Marble (30 to 150 N/m^2)
  - Metamorphic– Schist (5 to 60 N/mm^2)
  - Scatter in durability (In general stone is obtained from the upper part of the quarry = altered material)

- Clay brick in ancient masonry
  - Thickness of 4 to 7 cm
  - Other dimensions are much variable (22 × 11 cm^2???)
  - Large porosity (20-35 %)
  - Low strength (5 to 20 N/mm^2)
  - Low durability (hand made; burnt in a traditional wood / coal kiln)

- Adobe
  - Rather low strength (0.5 to 3 N/mm^2)

Mortars

- The use of mortar has the following purposes:
  - Bind the masonry units together
  - Reduce the effect of the irregularity of the units and make laying the units easier
- Mortars take time to harden and are usually weaker than masonry units
- Mortars can be made of mud, bitumen (Mesopotamia), gypsum (Egypt), (hydrated or aerial) lime, hydraulic lime and cement

- Addition of pozzolana or brick dust allows to create hydraulic products (i.e. capable of hardening under the water)
- It is usual that mortars contain silty soil (50 to 100% of the aggregate)
- Dry masonry (i.e. without mortar) is also usual

WALLS

Walls have several functional and structural roles:

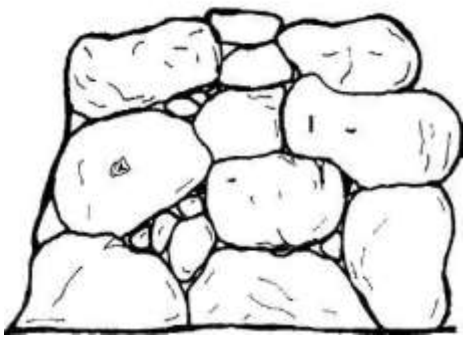
- **to form an envelope to provide shelter from sight, wind, rain and temperature**
- **to support the weight of floor and roof systems**
- **to provide in-plane strength and thus contribute to resist lateral forces (wind, earthquake)**

Most walls in historical construction are load bearing ones. In some structural systems non-load-bearing walls can exist. Even in these cases, they are normally still structural members providing buttressing or in-plane strength (shear wall).

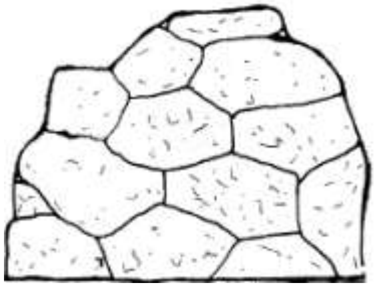
Some historical walls were provided with specific seismic resistant details, as iron connections between cramps (Greece) or geometric devices (Egypt, Peru)

58

WALL TYPES



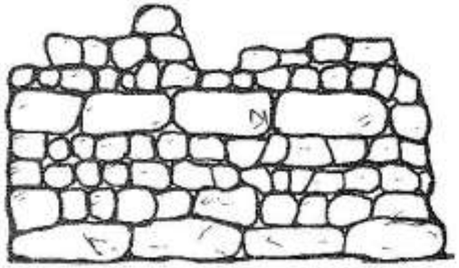
Rubble megalithic masonry with large irregular stones.



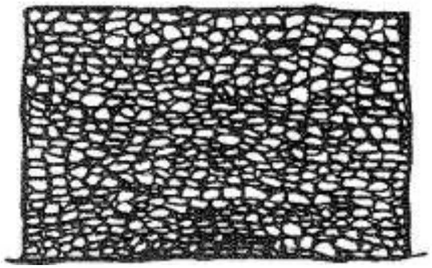
Polygonal stone masonry.

Square blocs placed w/o pattern.

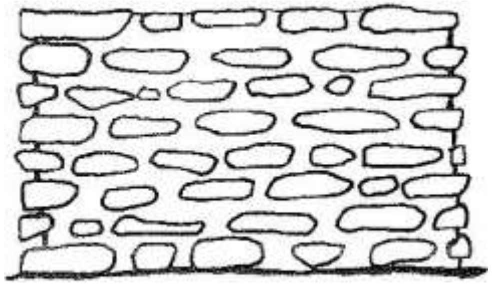
Square blocs placed with a defined pattern.



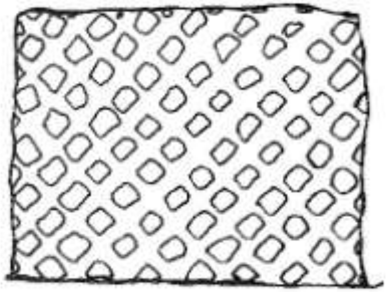
Opus squadratum



Opus cementitium.

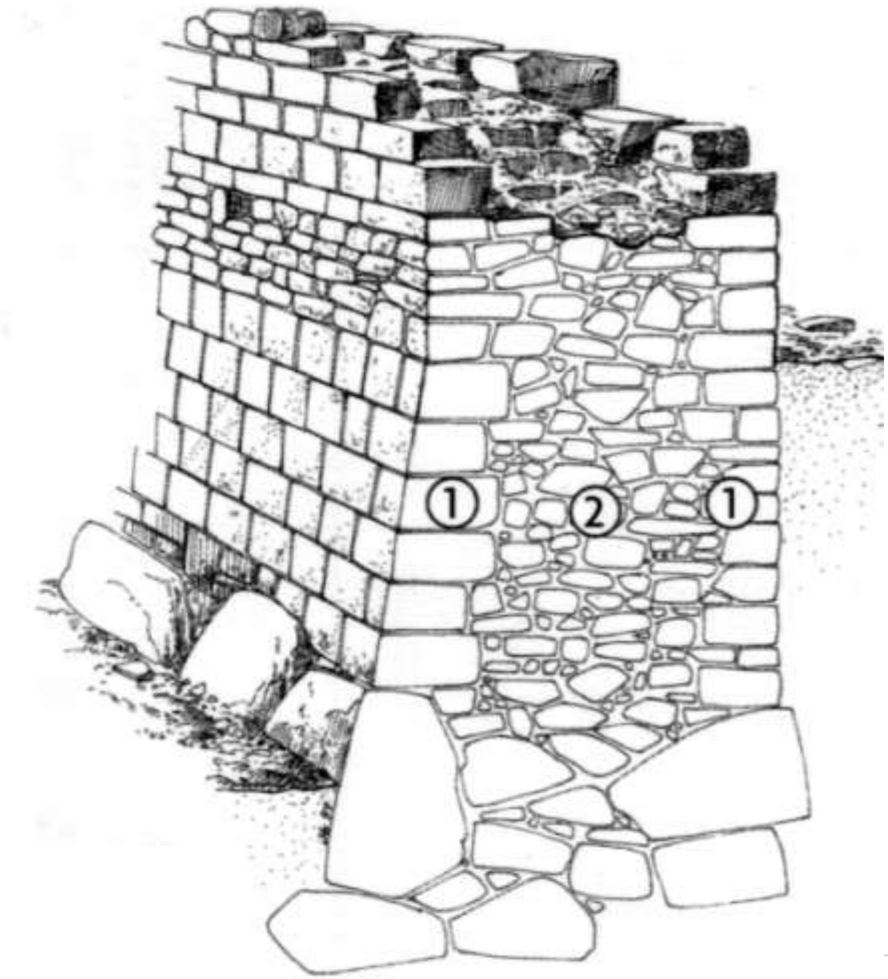


Opus lateritium.



Opus reticulatum.

WALL TYPES –HETEROGENEOUS WALLS



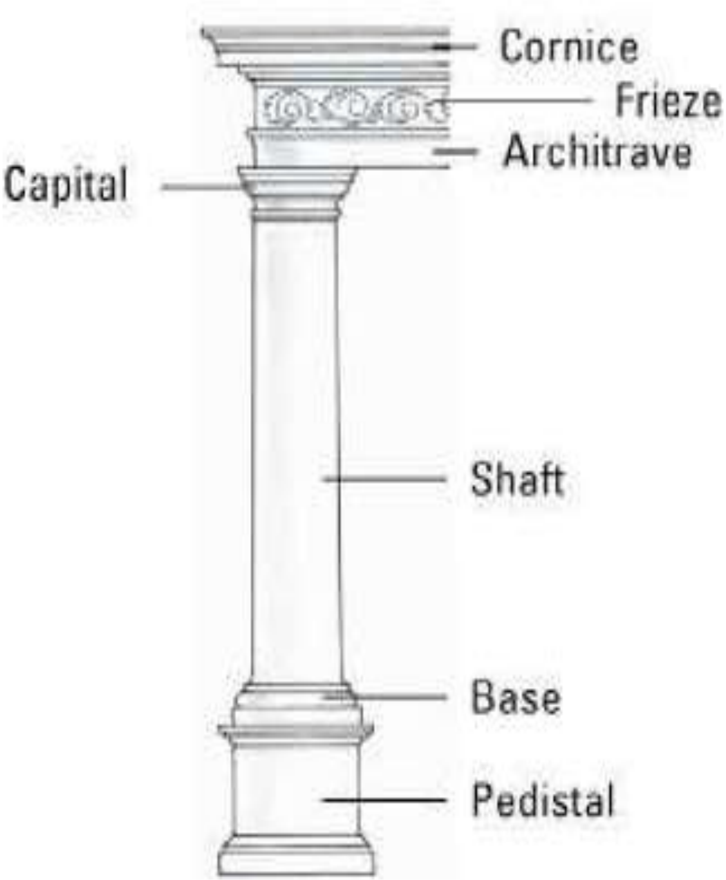
Three-leaf wall, used since the Roman period on. Cut stone or brick external leaves are filled with internal rubble masonry. Adequate connection (interlocking) between the exterior and interior leaves is essential to avoid leaf separation in the long term.

COLUMNS AND PIERS

Columns are vertical members to support concentrated loads transferred by arches or beam systems (architraves). Contrary to the walls, they hardly contribute to resists horizontal forces.

Large columns are normally built of large blocks (drums) or masonry. In some cases, full columns had been carved from a single rock block.

Columns are normally complemented with a basement and a capital. Some of them are tapered.



MEDIEVAL MASONRY PIERS

Masonry piers normally combine an exterior leaf of regular masonry with an interior core of stone or rubble filling.

In some cases, they are entirely composed of large blocks adequately interlocked, with no differentiated core.

ARCHES AND VAULTS

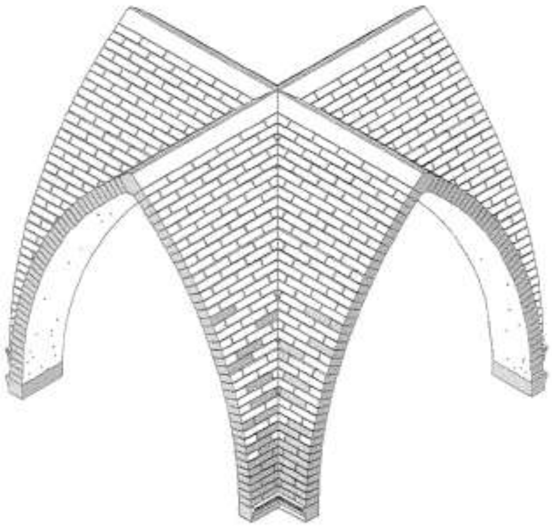
Arch and vaults construction provided, historically, the possibility of larger spans with limited amount of material, compared with more primitive technologies (as post-and-lintel construction).

However, arch and vault construction faced significant challenges, and builders struggled across history to overcome them. These challenges are:

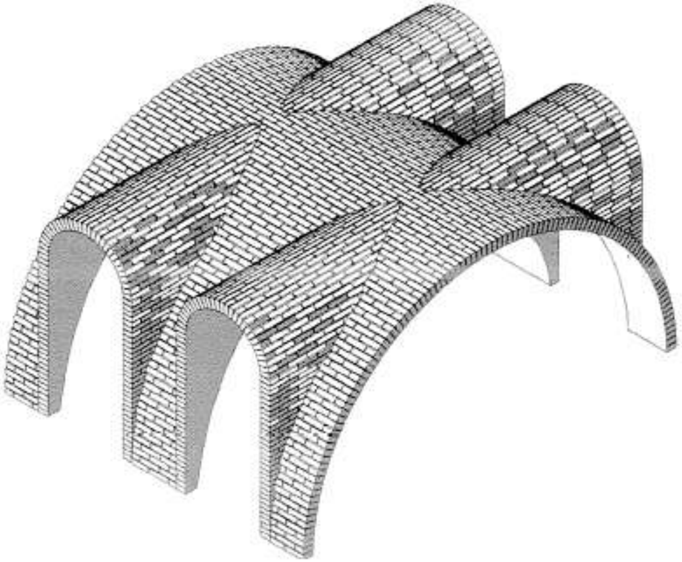
- **The need for centerings and forms (with significant consumption of material and work)**
- **Or, the search for arch shapes and techniques causing limited centering needs.**
- **The identification of shapes adequate to resistance**
- **The need for enough buttressing to counteract the lateral thrust caused by the arches**

63

Cross vault (Roman type) or groin (formed from intersection of two barrel vaults)



Gothic cross vault



Penetrated barrel vault or underpitch (a barrel vault with small perpendicular vaults underneath)

Fan vault (a group of ribs springing from a point to form a vault)

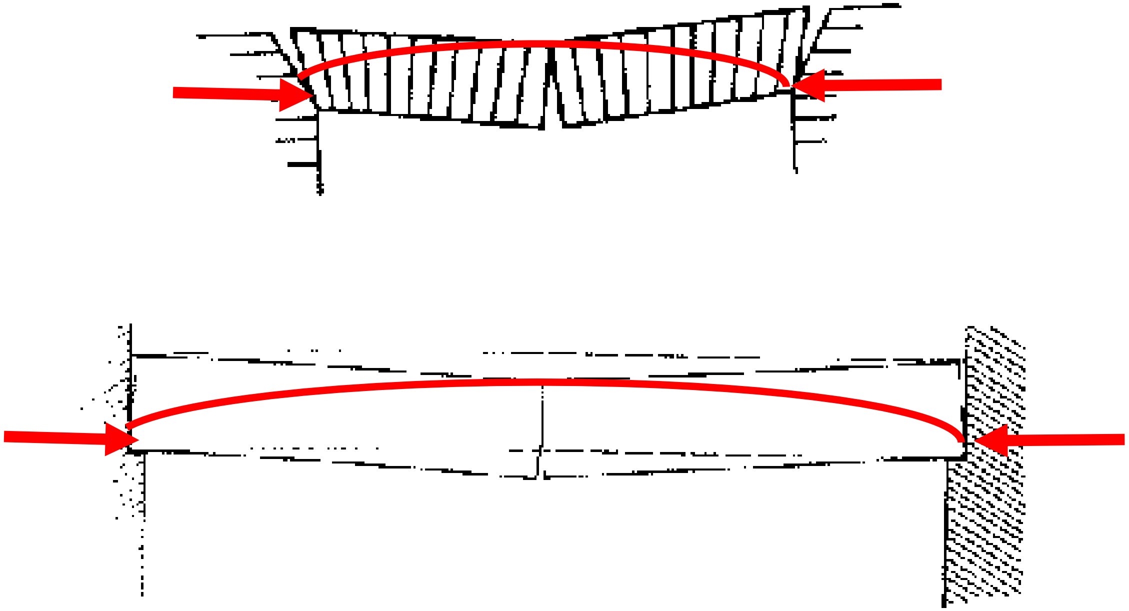
STRUCTURAL SYSTEMS

Historically, masonry structures have been conceived upon two alternatives:

Lintel construction (inspired by timber construction?) is based on the combination of pillars/walls and lintels, the latter consisting of monolithic stones able to resist some flexural forces. Their capacity is owed to stone flexural (tensile) strength.

Arched or vaulted construction (inspired by natural arches and caves?) , where the design is conceived so that stability is possible by only activating compression forces. Their strength stems from geometry.

Lintel construction – Greek temples are possibly inspired by primitive timber structures. They seem to retain the organization and resisting principles of timber-based construction.



Do lintels in Greek temples work, in fact, as jack arches? This seems plausible as many lintels show cracks at mid-span (hinges) and still keep stable. The arch-like work and the lateral thrusts is activated as soon as the lintel cracks due to initial bending forces.

…and more recent architectural approaches as Mongol (Mughal) construction in India (Red Fort, Agra, built 1565-1573 AD.).

Fatehpur Sikri, built ca. 1569, obviously inspired by (and even mimicking) wooden construction arrangement and details

Lintel construction is confined to very small spans due to the need to limit the bending forces experienced by the stone.

In fact, the stone's tensile strength is a very delicate property which can easily deteriorate due to a variety of actions (earthquake, thermal variations, settlements, chemical attack, erosion…)

Weren't for the possibility of working as jack-arches (and the supporting structure being robust enough as to counteract the resulting horizontal thrusts), almost all post-and-lintel ancient structures would have collapsed.

71

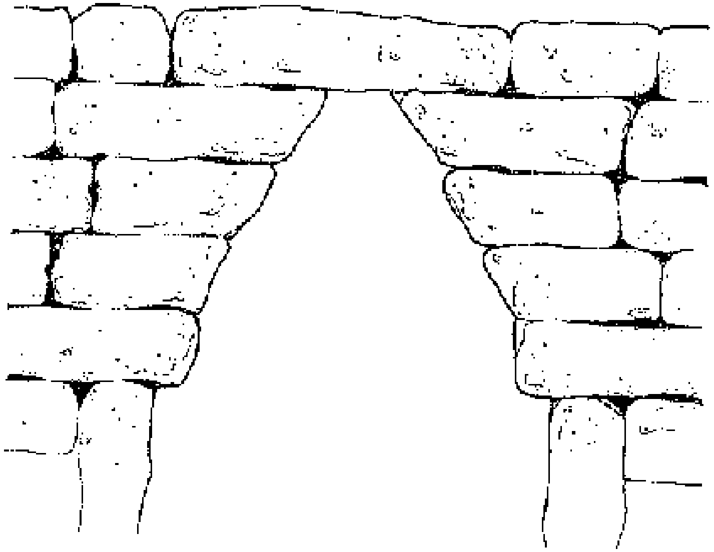
VAULTED CONSTRUCTION

A second possibility, suggested by nature itself, comes after the use of shapes which are stable by only mobilizing compressive forces – arches and vaults. Used already by ancient civilizations in Mesopotamia, it was intensively exploited by Romans and has prevailed as the main roofing approach for large constructions up to the 19^{th}-20^{th} c. technological revolution.

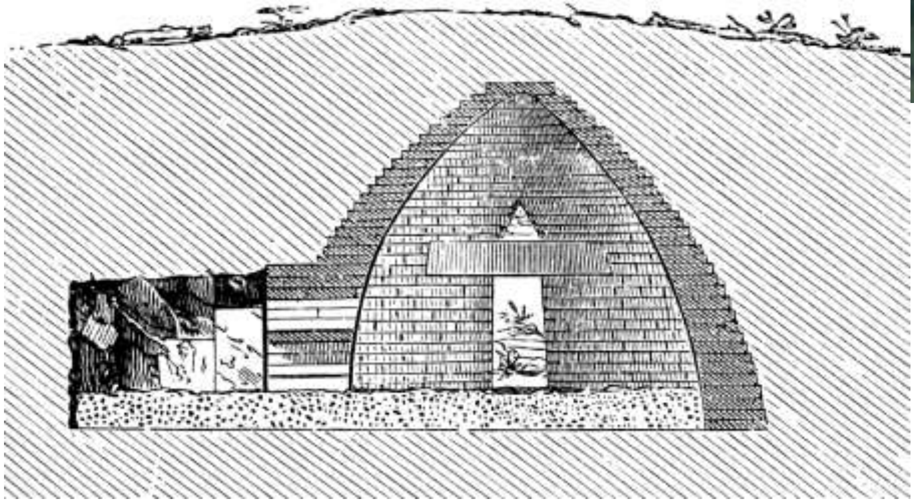
72

FALSE VAULTING

True vaulting poses significant construction challenges, as the need for expensive and time-demanding shores and centering. Some cultures (Greek Mycenaean, Sumerian, Sasanian Persia, Pre-Columbian Mesoamerican, Khmer) skipped these difficulties by resorting to false vaults and arches (or corbel, corbelled -) built as sequence of self-stable cantilevers. The technique affords only very limited spans to the cost of a significant rise and high consumption of material.



Lions's Gate, Citadel of Mycenae, Greece 2nd millennium BC

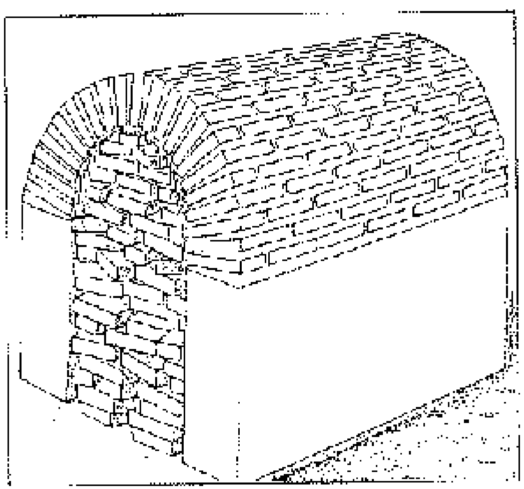
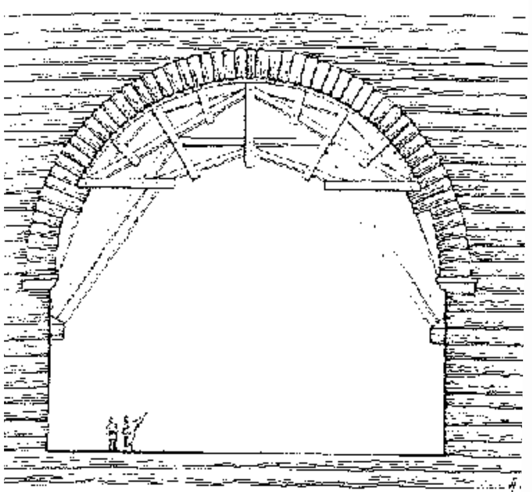
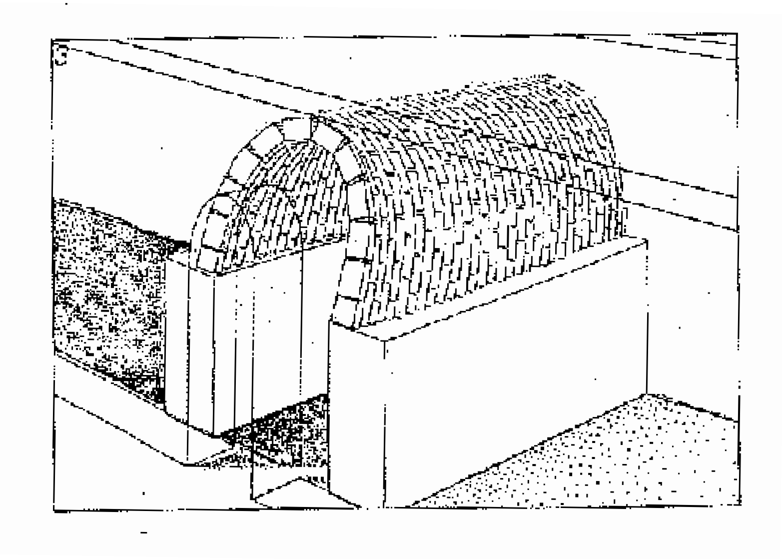


Tomb of Clymnestra (above) and Treasury of Atreus, below, typical Tholoi tombs of Mycenae, 2nd millennium BC

74

True arch and vault construction has been achieved by the following means

- **Using earth or rubble fillings and mounds instead of centering**
- **Use true centering and shores (normally made of wood)**
- **Using smart construction procedures avoiding or reducing the need for centering**

75

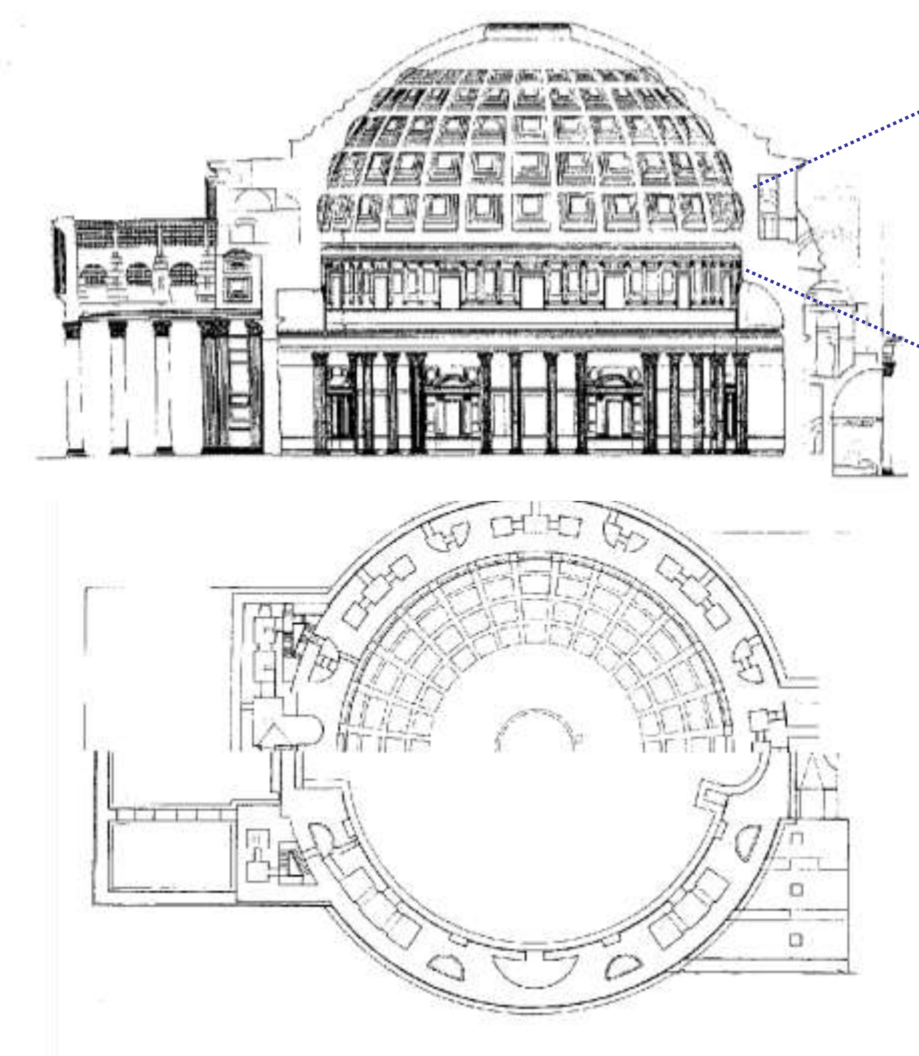
ROMAN CONSTRUCTION

Romans used intensively the possibilities of arches and vaults. Their more outstanding constructions included large-span concrete roofs (up to 20 m for vaults and 40 m for domes) using a variety of solutions (barrel vaults, cross-vaults and domes)

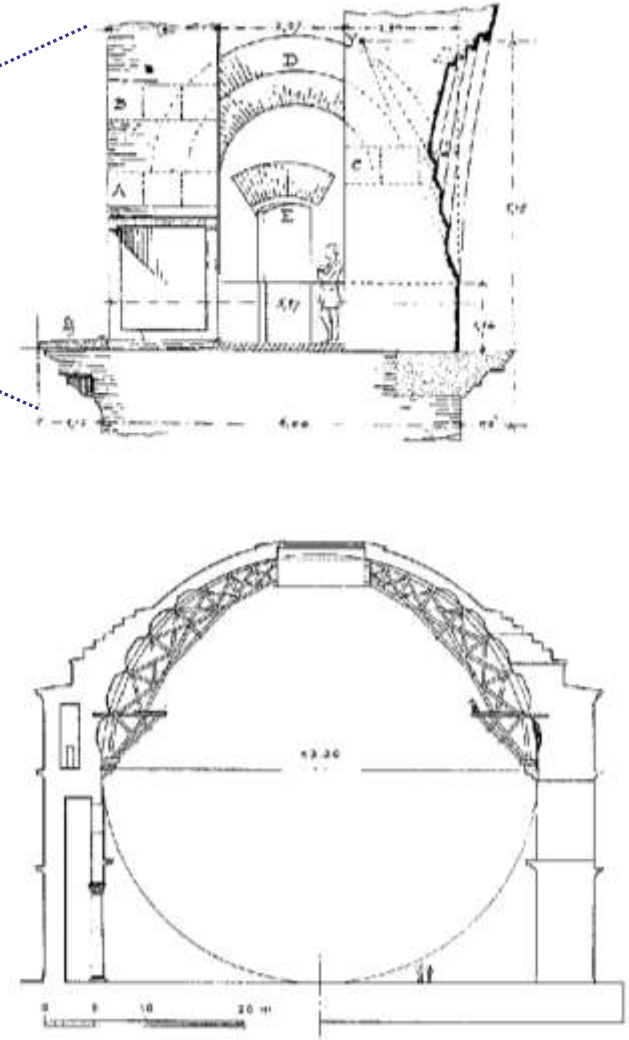
A large part of their construction was in pozzolanic concrete, which was easily adaptable to curved shapes. However, the resulting vaults were very delicate and difficult to repair. Unreinforced concrete cracking would result in very inconvenient aesthetic, maintenance and, eventually, structural problems.

To avoid cracking in concrete vaults and domes, these had to be supported on massive walls and foundations.

Pantheon, dating from c. 125 A.D., consisting of a dome of 43.3 of diameter, 43,3 m high at the oculus, over a 6.4 m thick drum wall.

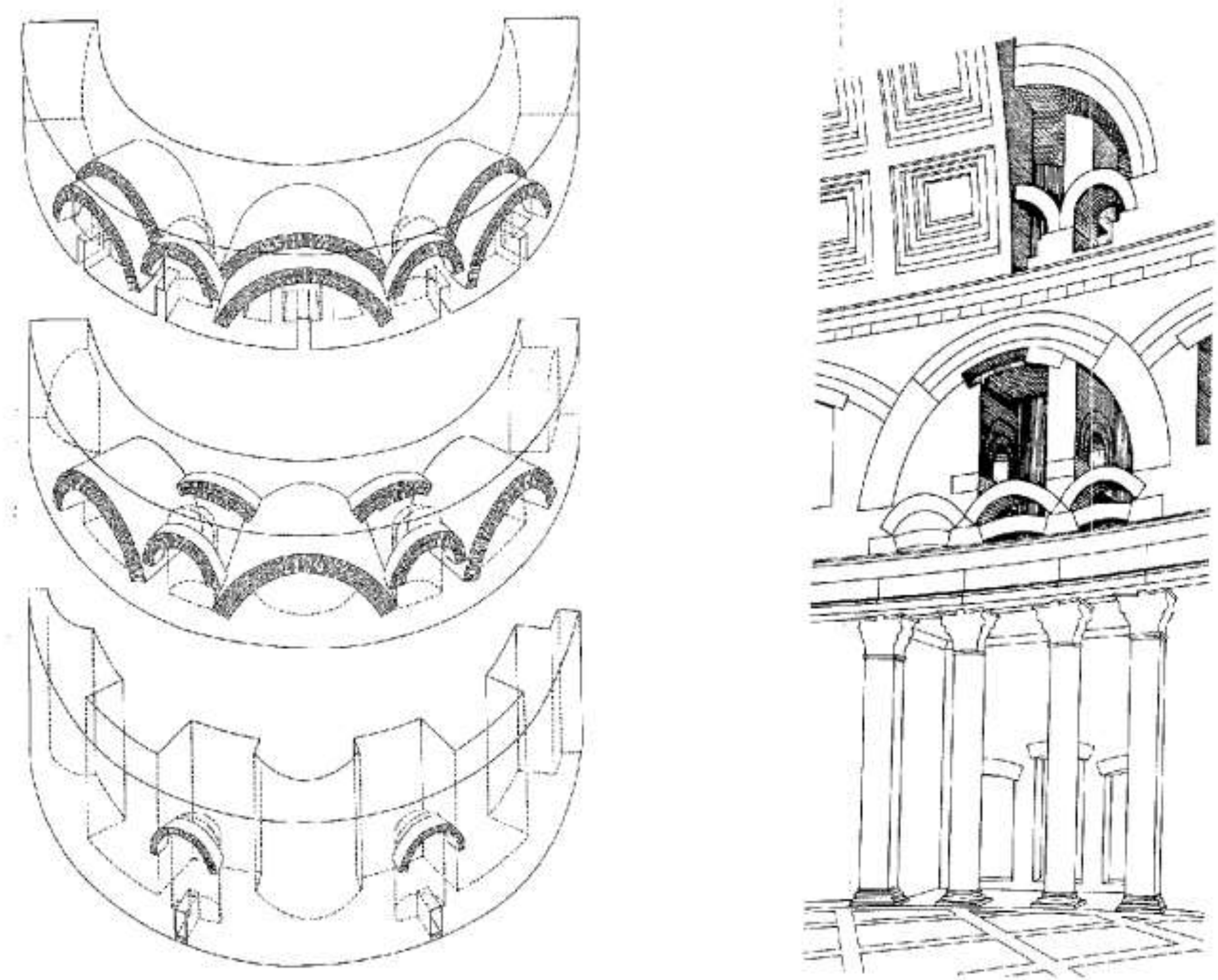


Section and plan



Relieving arches (above) and centering hypothesis (below)

The loss of the stone veneer permits recognize the concrete cylinder as an organized structure including a system of brick relieving arches.

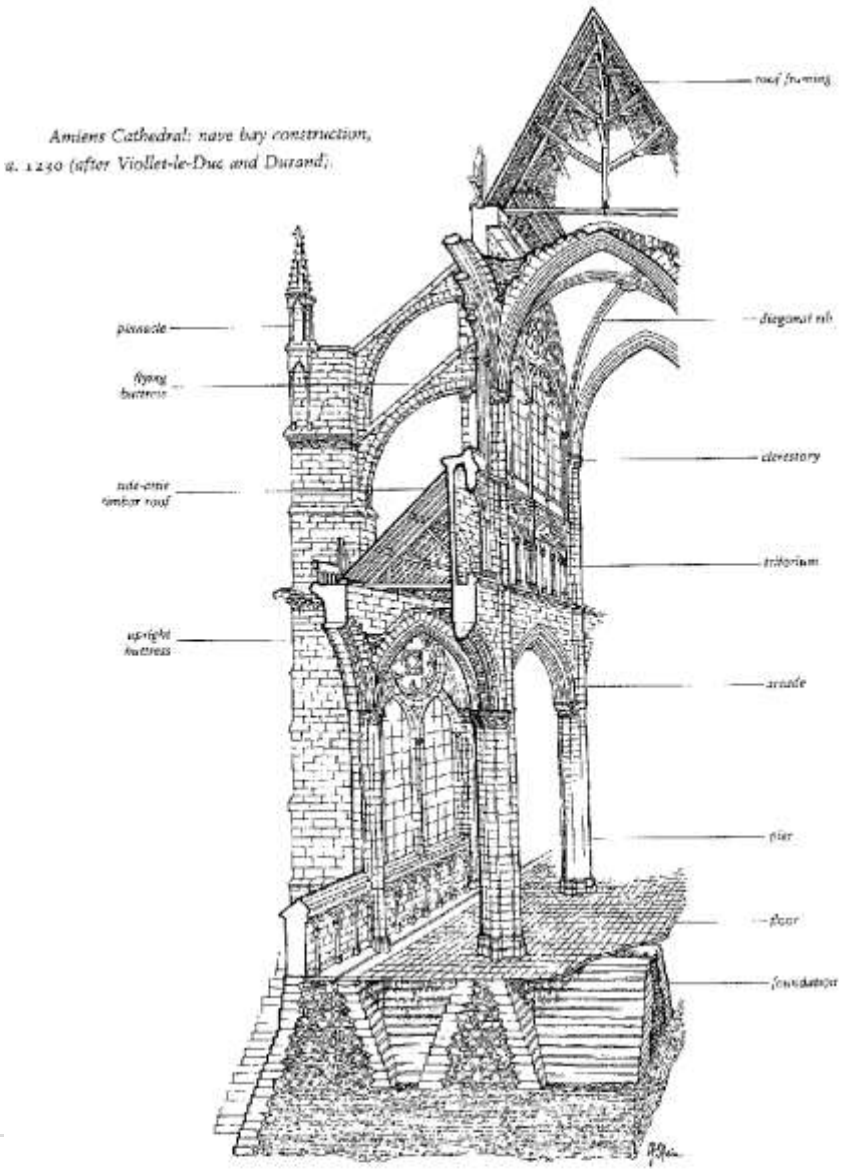


System of brick masonry relieving arches embedded in the cylindrical walls

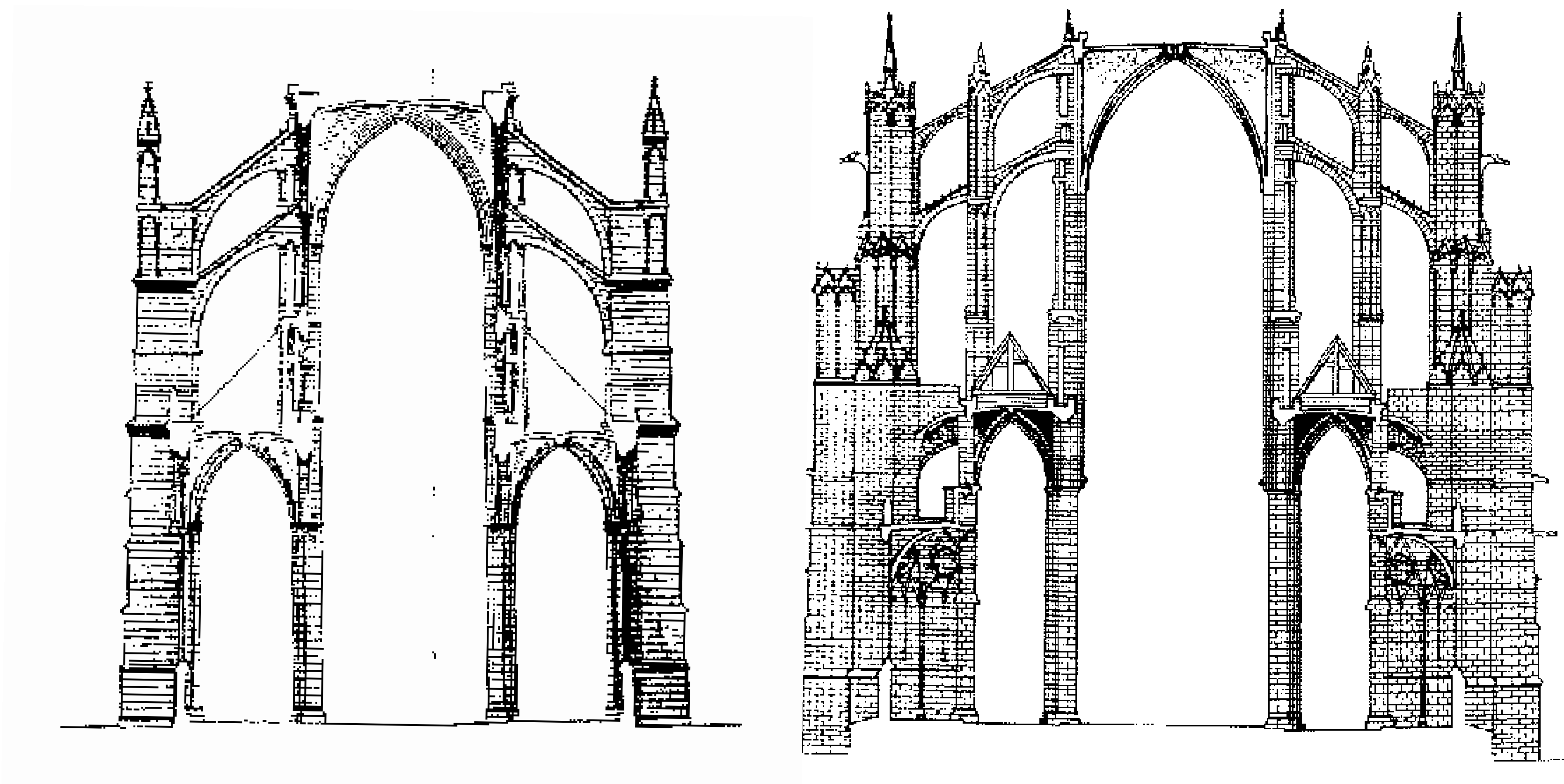
80

GOTHIC CONSTRUCTION

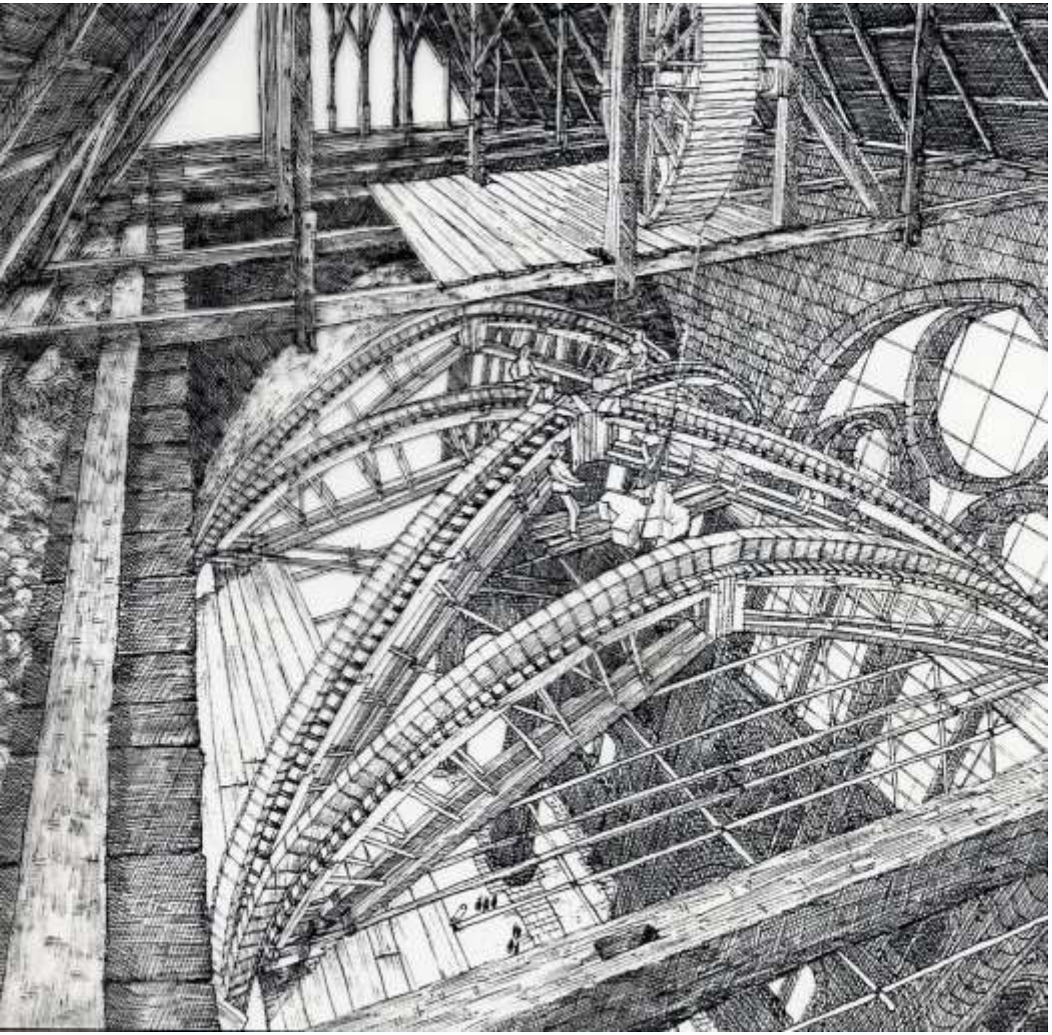
Gothic construction, made possible by the advent of more sophisticated construction techniques, produces truly skeletal structures composed of arches, nerves, flying- arches, piers and buttresses. No structural 2D members exist, except for the membranes spanning across the nervures.
All typical Gothic structural members had been already used by former architectural cultures (flying arches by Byzantium, cross-vaults by Rome and former Medieval architecture...). The specificity of Gothic architecture is in the way these members are combined to lay-out a pure skeletal structure where forces are adequately balanced and neatly channelled towards the buttresses and foundation with close to minimum material consumption.
This accurate adjustment affords for significant material saving, structural slenderness and clearance (compared with other architectural approaches).



Transverse section of Amiens Cathedral according to Viollet-le-Duc



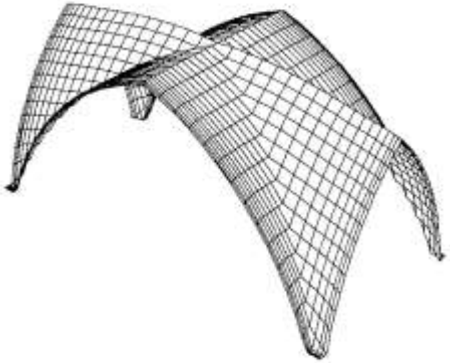
French High Gothic outstanding examples: Transverse section of Amiens cathedral (begun 1220) and Beauvais cathedral (choir, begun 1225). Maximum vault's height of 42 and 48 m).

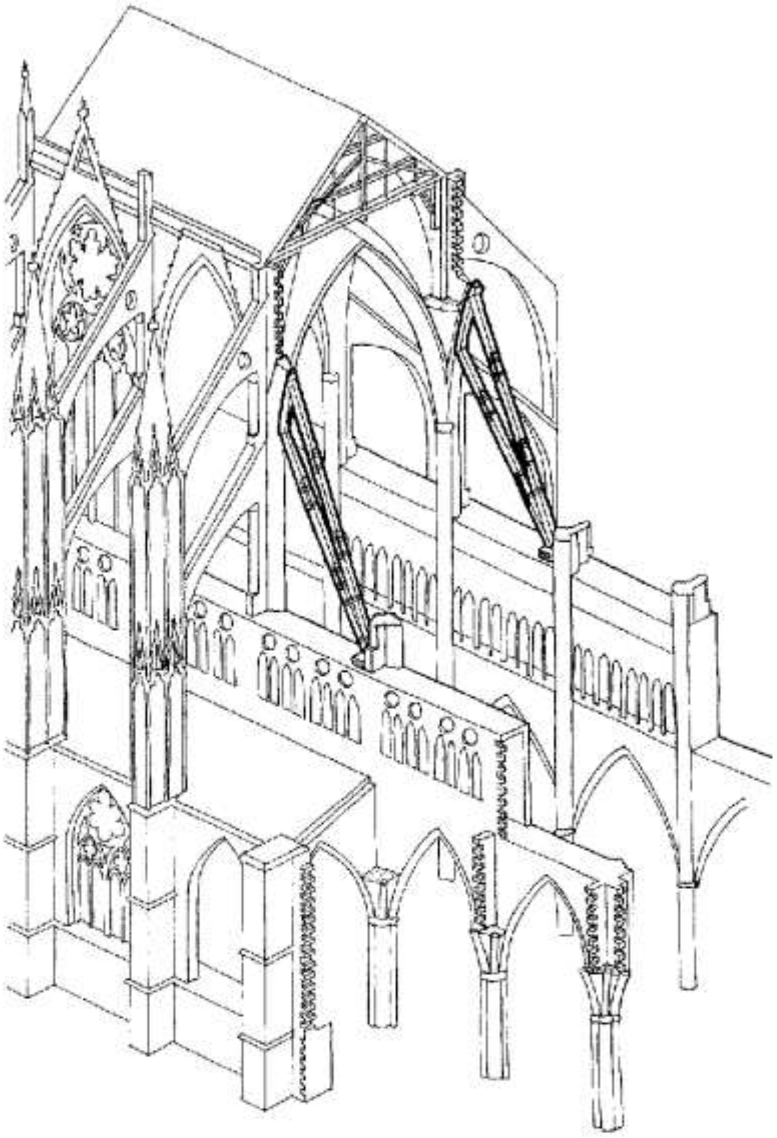


The success of Gothic architecture owes very much to the Gothic cross vault, whose construction only required to use centering for the arches and nervures.

The Gothic cross-vault does not comply with any mathematical equation – it is a free shape resulting from the construction procedure.

As opposite to domes (without confinement), Gothic vaults do not systematically crack.





The fact that the structure reaches equilibrium (and thrust balance) only in its complete configuration, may have generated significant difficulties during the construction.

Left: Unlikely provisory construction arrangement suggested by Viollet-le-Duc.

Delicate construction stages might have had to be overcome. Significant temporary structures or devices may have been needed. Some existing damage and deformation (as in Mallorca Cathedral) may have been caused during the construction process.
In many cases (Barcelona Cathedral...) the Gothic structure replaced a former Romanesque church whose remaining walls, while gradually demolished, were used as provisory buttresses.

Mallorca Cathedral (14-15th c.) was considered by Robert Mark as the epitome of Gothic construction due to its unique combination of height over ground (44 m) and clearance (the central nave vaults span 19,4m). The extreme slenderness of the piers (height/diameter = 1:13-1:15) can only be compared with that of Jeronimos Church in Lisbon.

THE CONSTRUCTION OF DOMES IN EUROPE DURING 15-16th c.

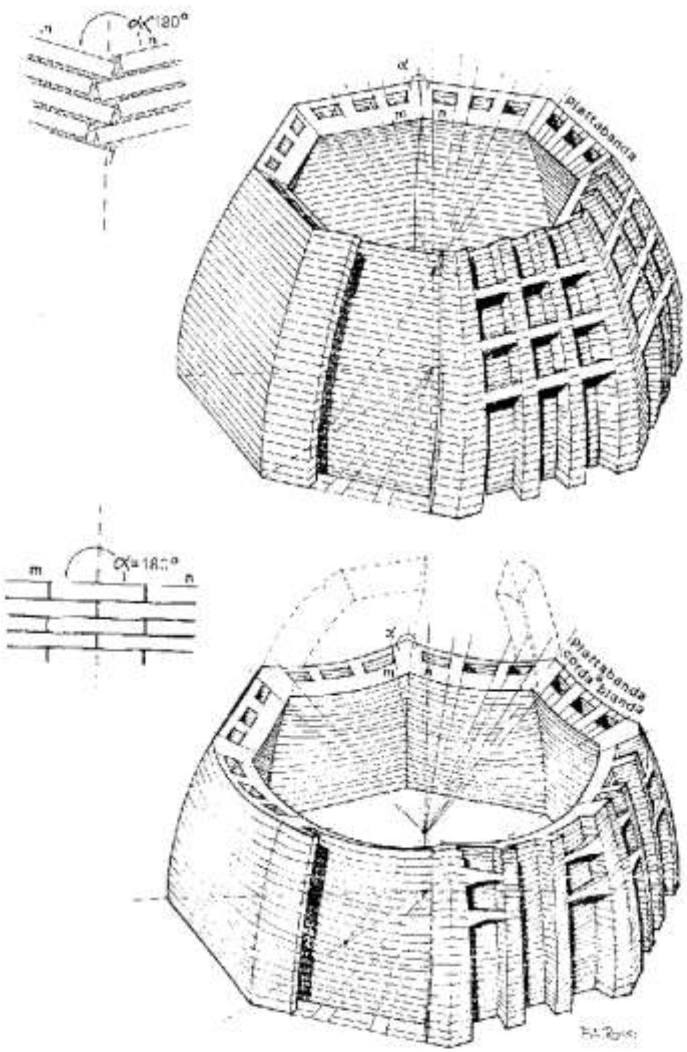
Renaissance in Europe brought a recovered interest in ancient classical (Greek and Roman) architecture. The dome was recovered as outstanding roofing solution for emblematic buildings, while Gothic architecture was regarded as contrary to classic conception and even despised.

In spite of the philosophical rejection of Gothic architecture, many Gothic architectural resources prevailed due to their rationality and optimality. In fact, Renaissance architecture constitutes a synthesis of classic and Gothic, rather than a mere recovery of the first.

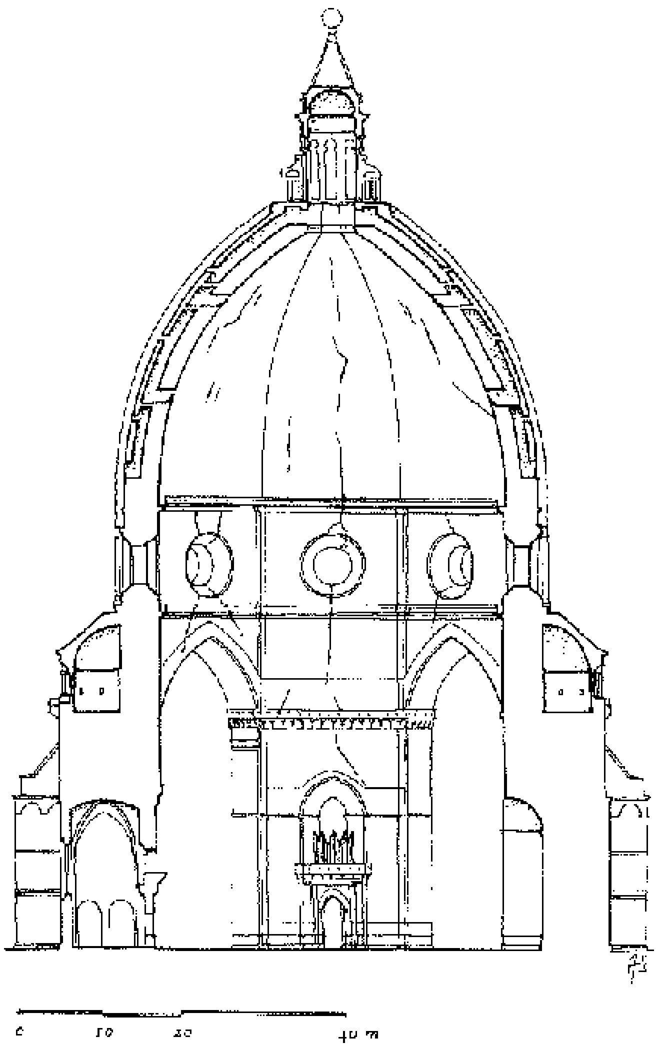
This synthesis can be clearly identified in Brunelleschi's dome in Florence, which includes remarkable Gothic treats such as the pointed geometry (to reduce the horizontal thrust) and the ribbed (and certainly complex) membrane supported on meridian arches. These combine with the concept and dimensions of the dome itself, taken from the Pantheon.

Filippo Brunelleschi's ingenuity produced one of the most complex structures ever built and still not totally understood – the dome of Santa Maria del Fiore in Florence (built 1419-1436, lantern built 1445-1461)

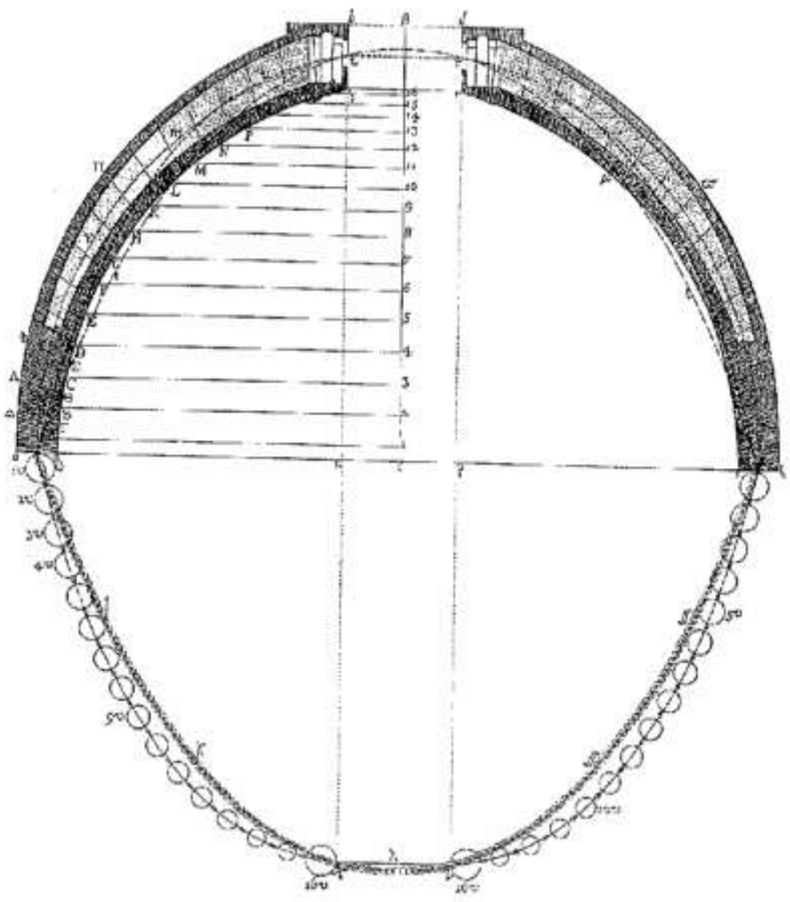
The purpose – allowing the construction of a Pantheon-like dome over an existing masonry structure 54 high, without resorting to any centering or forms – was successfully accomplished. The dome has a diameter of 43,6 m and a rise (interior) of 33 m. The total weight of the dome is of 37.000 metric tons (!).



Complex building features



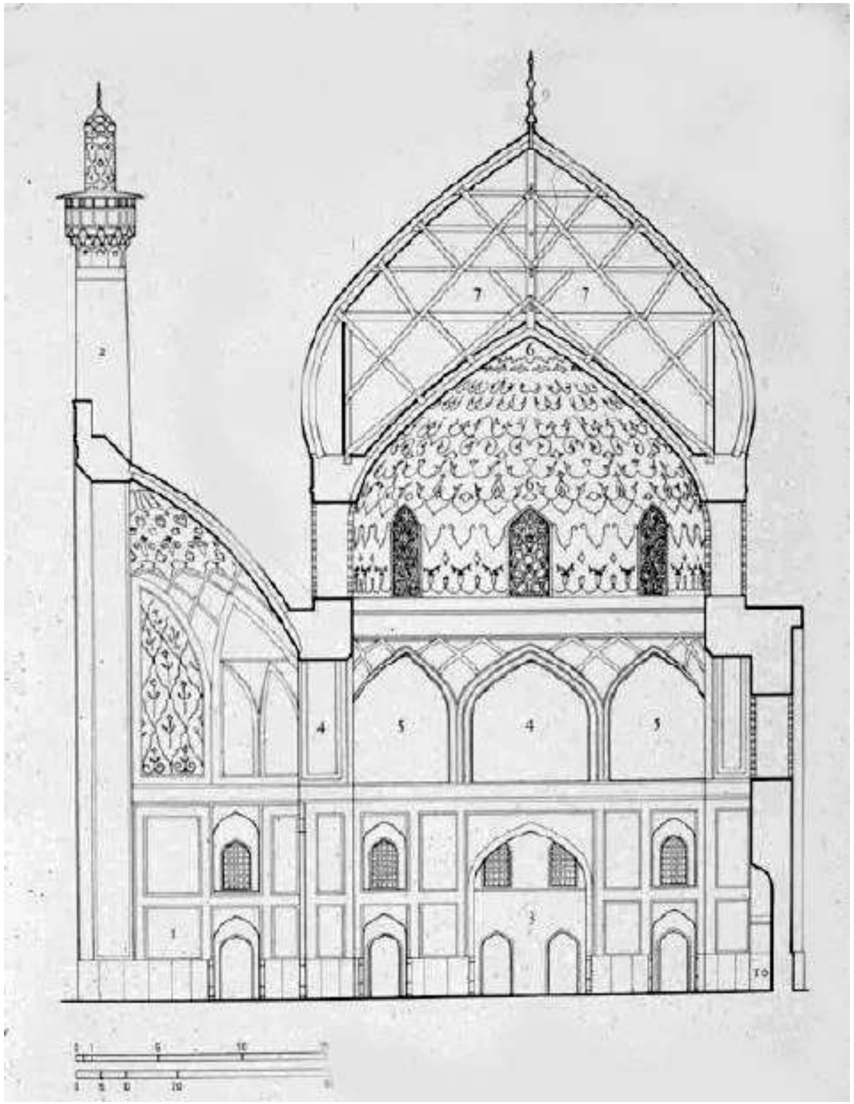
Today's cracked condition



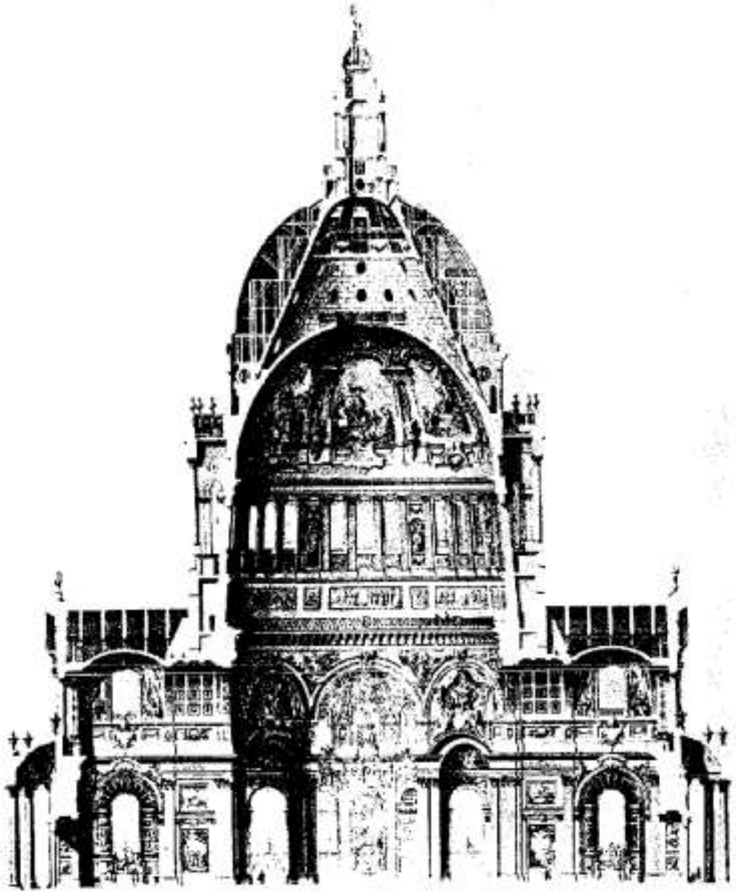
In 1743, Giovanni Poleni, an Italian mathematician, proved Saint Peter's dome full adequate design by applying the catenary principle.

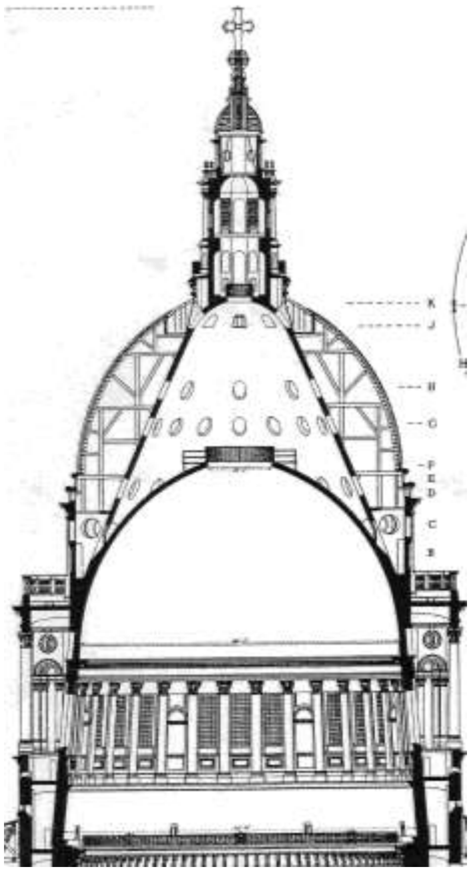
Poleni showed that the radial cracks (existing in almost all domes) were by no means connected to problems associated to collapse.

However, Poleni proposed to strengthen the dome by placing a set of iron rings. The rings, six in all, were fixed on the dome between 1743 and 1748.



Evolution of Persian dome construction, and the interest for complex and appealing external shapes, laid to the conception of sophisticated systems consisting on timber structures (providing the external volume) supported over pitched structural inner domes.





Hooke’s structural insights lead to a new and more economical way of producing an emblematic dome: the true structure consists of an almost conical dome (designed by Hooke), whose shape is made adequate by the above 8500 kN lantern. Lighter domes, with little structural role, are below and above the cone (the latter as a wooden structure supported on the dome).

המועצה לשימור
אתרי מורשת בישראל

Models for Masonry Structures

isise

Universidade do Minho

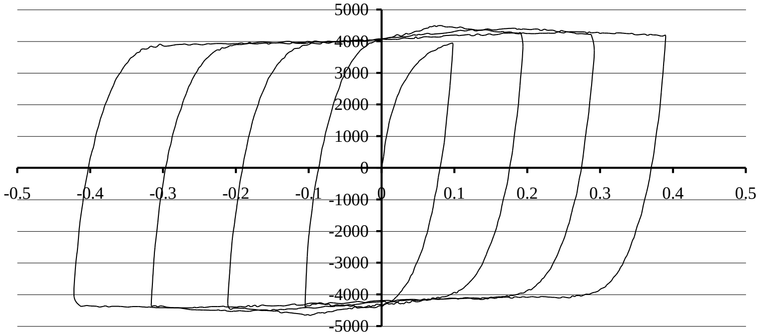
What is masonry?

What is masonry?

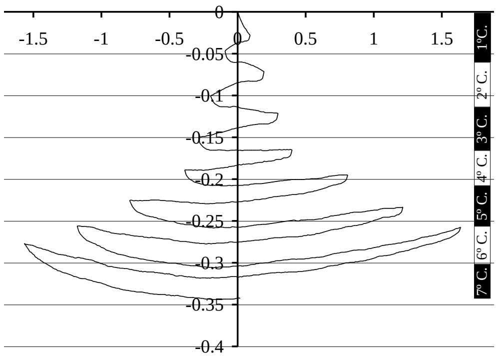
Masonry can be defined as a material with visible internal structure

Why is this relevant for mechanics?

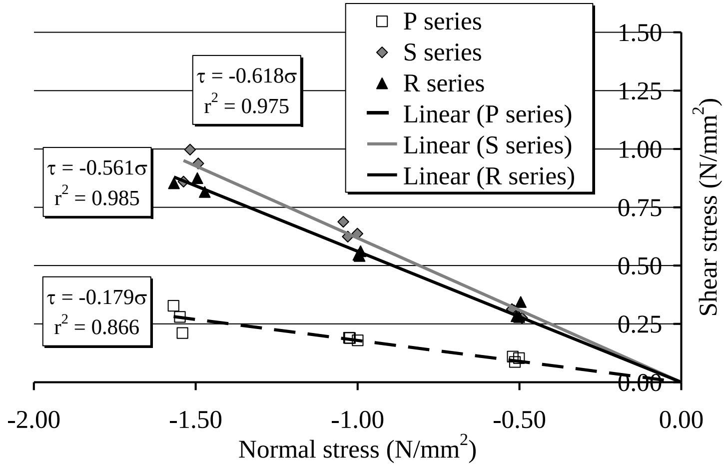
Shear testing of stone joints



Stress-strain relation

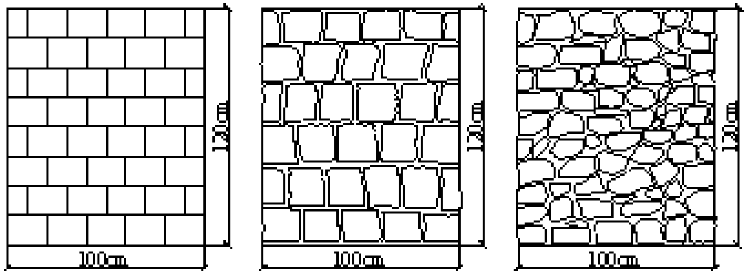


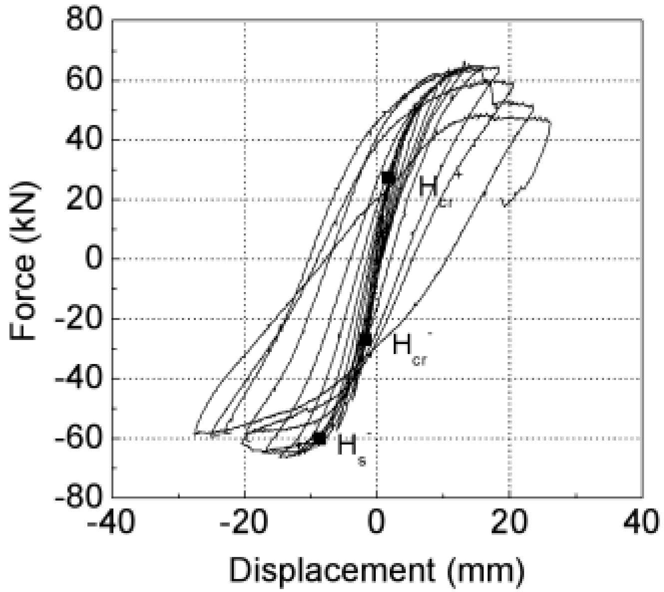
Dilatancy



Failure surface

Why is this relevant for mechanics?

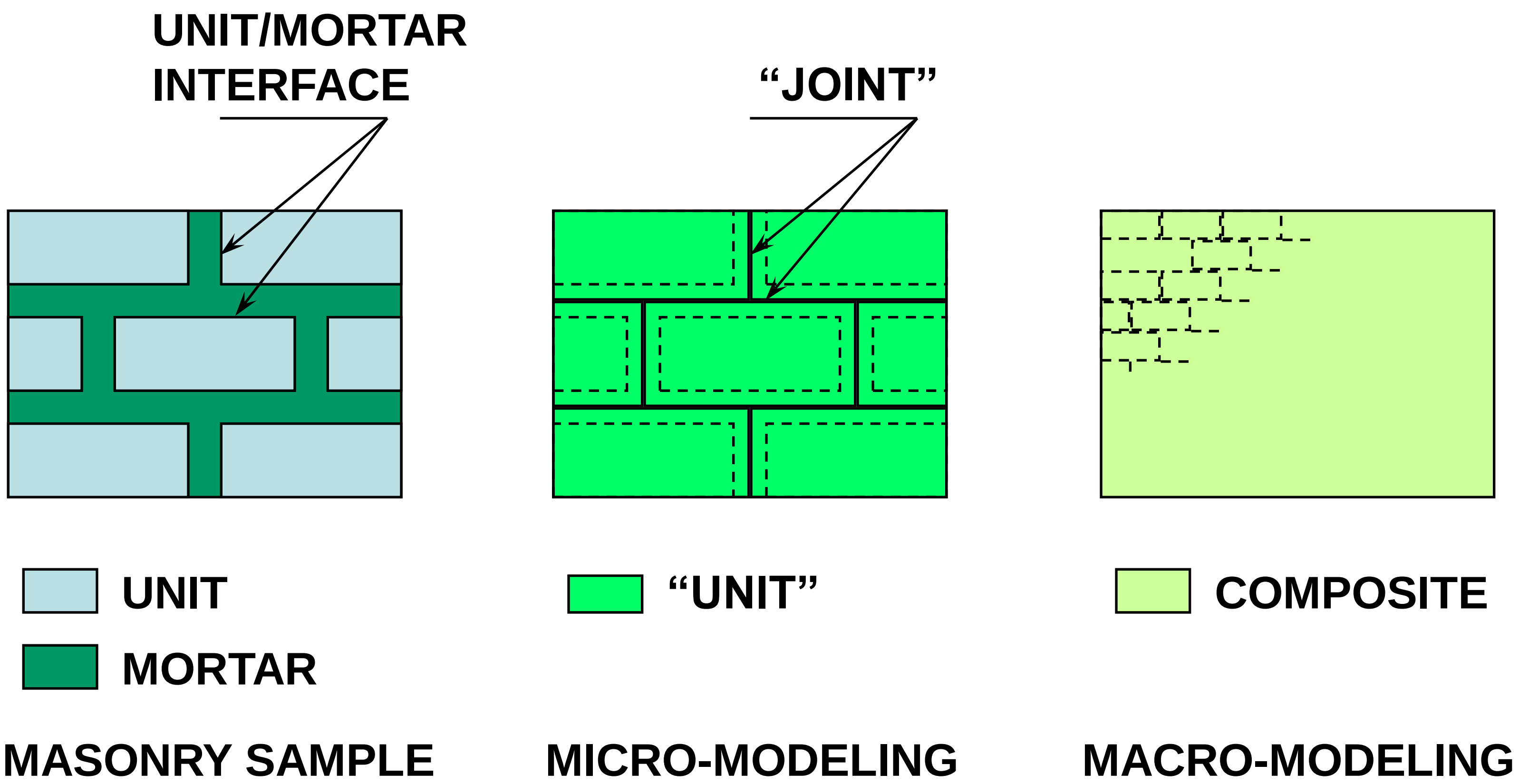




Stone walls

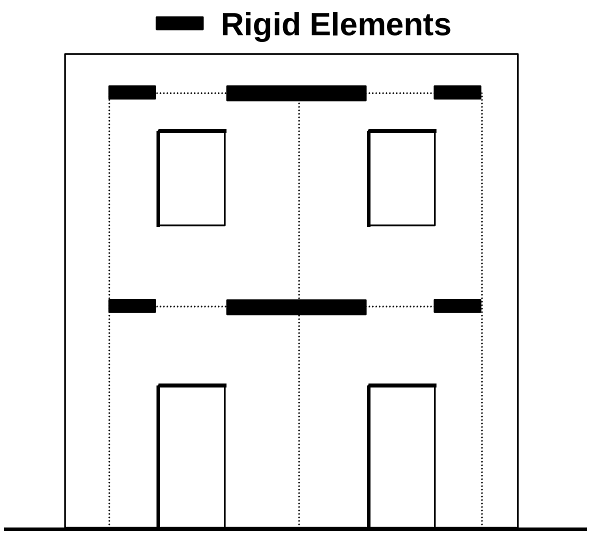
Collapse Mechanism and Strength
Regular – tanϕ = 0.4
Irregular – tanϕ = 0.3
Rubble – tanϕ = 0.2

Modeling masonry – Material Level

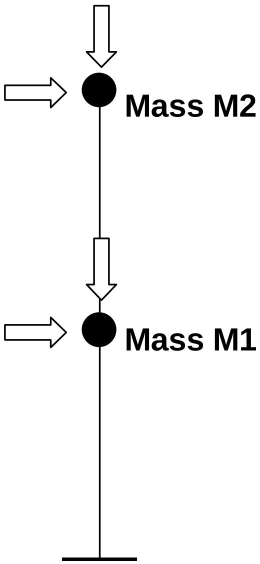


Sophisticated models require advanced material characterization. Much experience had been gained in the last decade (see paper)

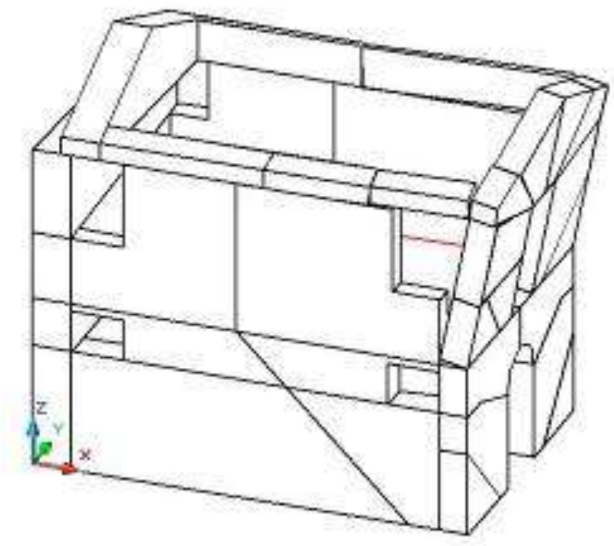
Modeling Approaches – Structural Level (I)



Beam elements model



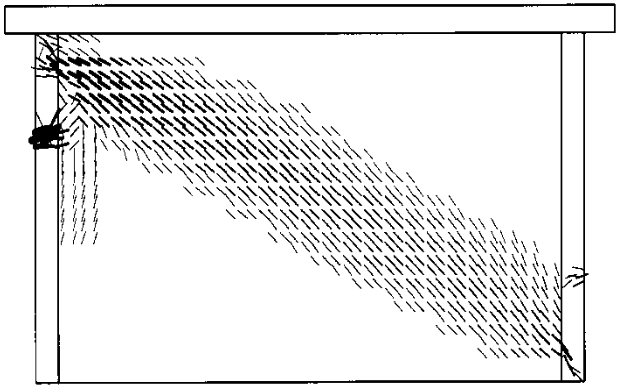
3-degree of freedom model



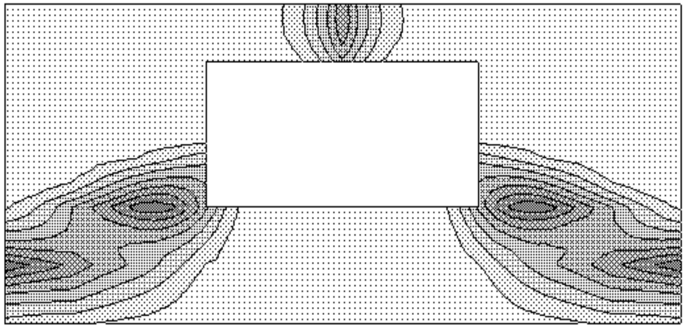
Macro-block model

STRUCTURAL COMPONENT MODELS

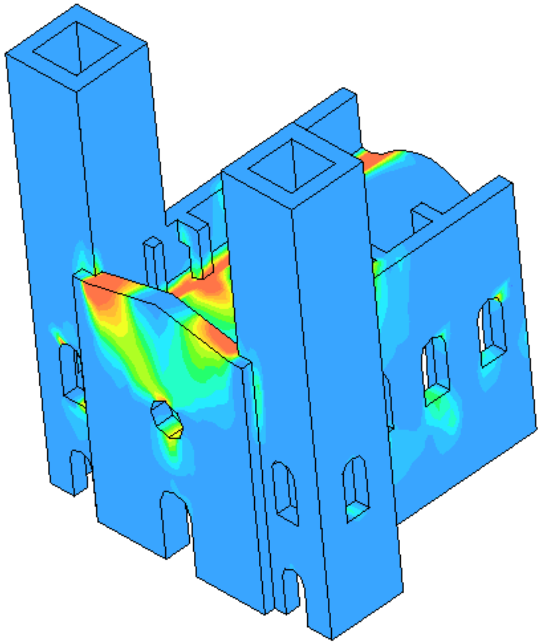
Modeling Approaches – Structural Level (II)



Shear wall



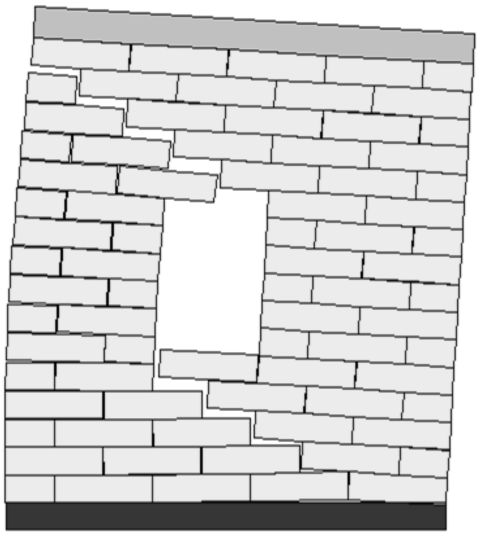
Wall with out of plane behavior



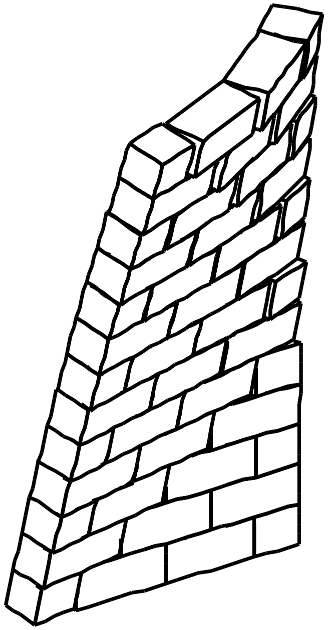
Church Settlements

STRUCTURAL MACRO-MODELING / FINITE ELEMENT METHOD

Modeling Approaches – Structural Level (III)



Shear wall
(in plane behavior)



Wall with
out of plane behavior

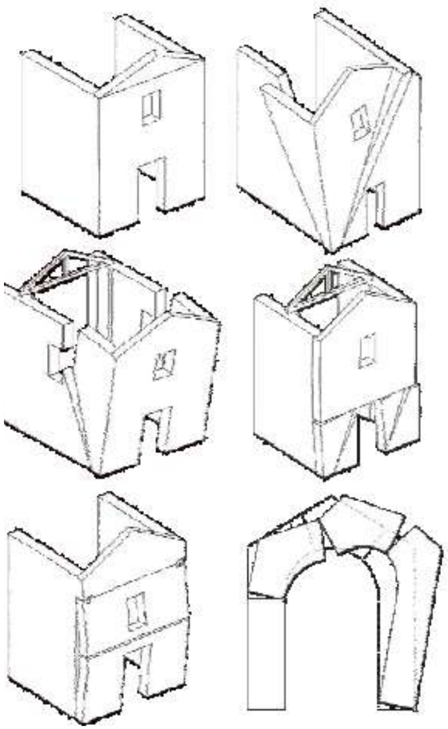
STRUCTURAL MICRO-MODELING / FEM, DEM, LIMIT ANALYSIS

Macro-Block Analysis

In the existing masonry buildings often partial collapses happen for seismic causes, generally for lose of the equilibrium in masonry portions; the verification for these mechanisms has a meaning if a certain monolithic behavior is guaranteed, so as point collapses for masonry decomposition is avoided.

The local mechanisms are of two types:

- Mechanisms for actions perpendicular to the plane of the wall
- Mechanisms for actions in the plane of the wall



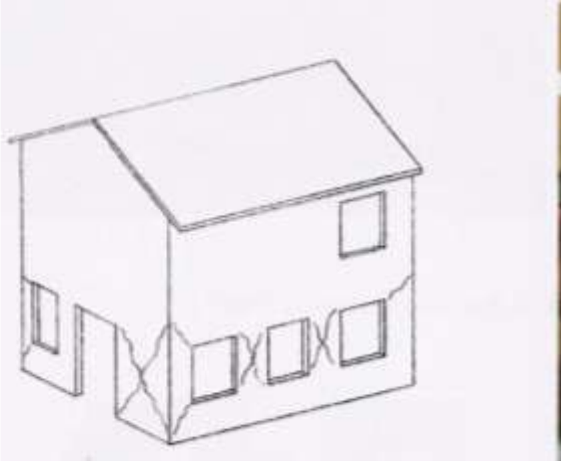
The verifications can be developed through the limit equilibrium analysis, following the kinematic approach (**principle of the virtual work**), that is based on:

1. the choice of the **collapse mechanism**
2. the following evaluation of the horizontal action that activates this kinematism

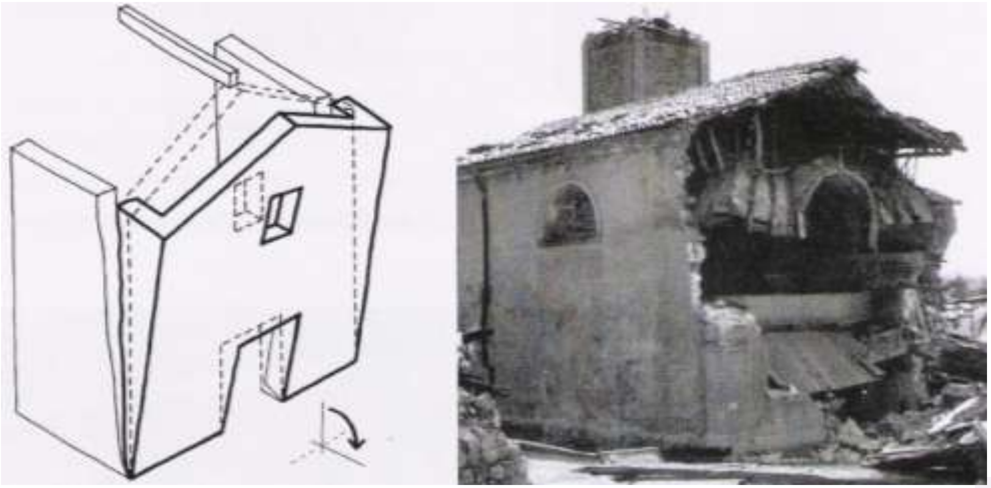
Collapse Mechanisms (I)

Shear

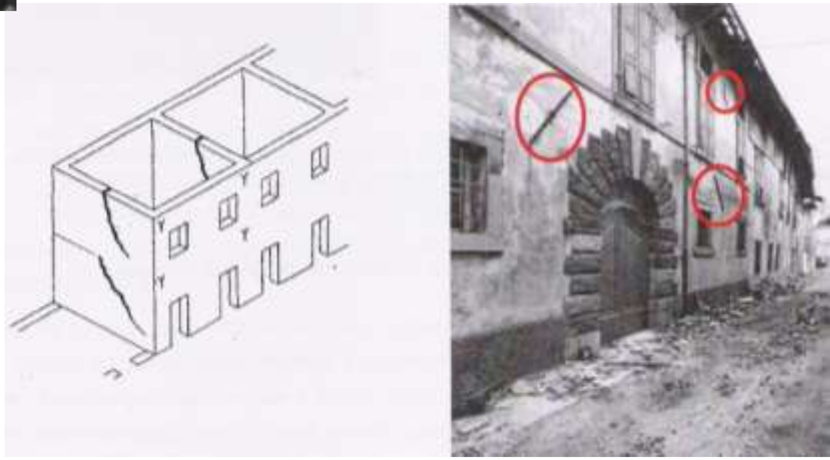
Diagonal tension



Collapse Mechanisms (II)



Overturning

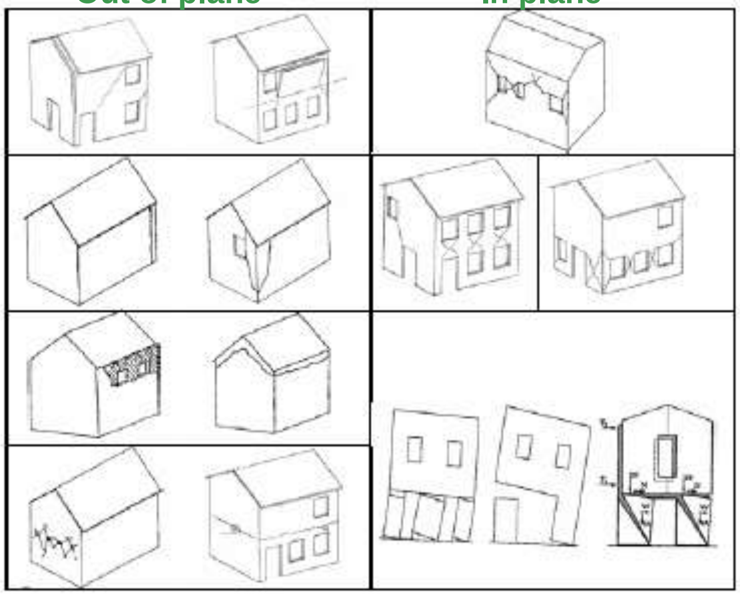


Overturning, limited due to the connection with transverse walls

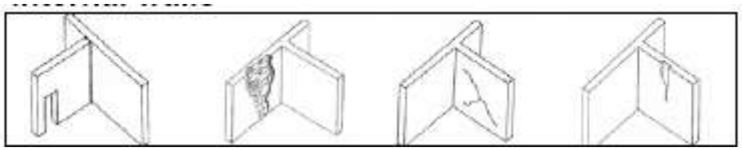
Abacus for buildings

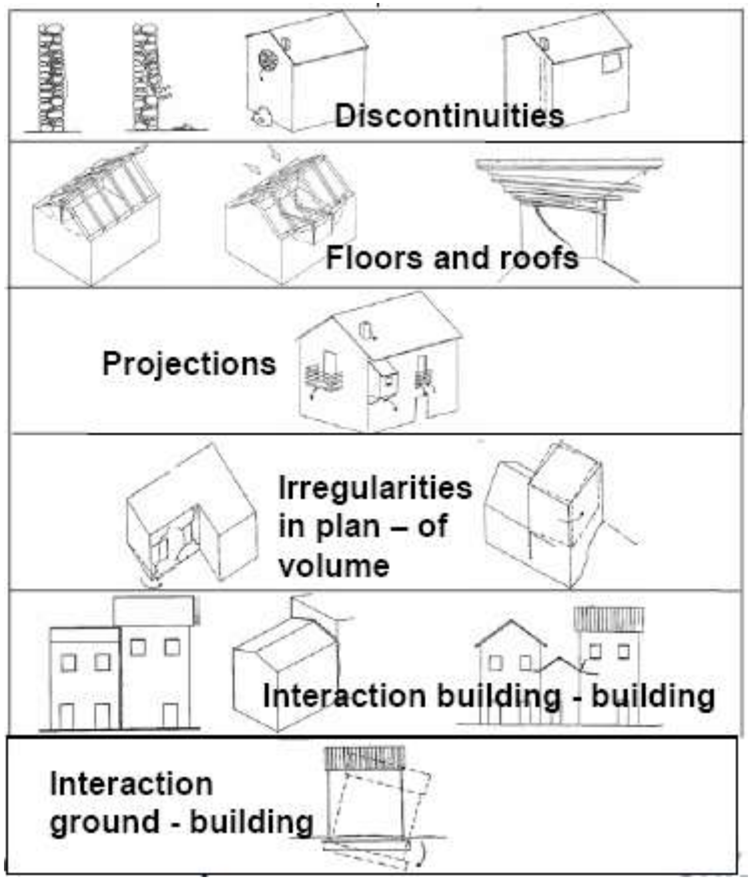
External walls

Out of plane In plane



Internal walls



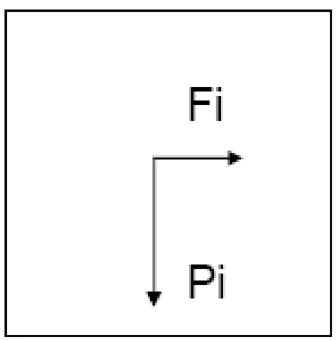


The Kinematic Approach

The kinematic approach to the problem allows therefore:

1. The evaluation of the horizontal action that activates the kinematism
2. The determination of the process of the horizontal action that the structure is progressively able to stand with the evolution of the mechanism, until the annulment of the horizontal force itself (i.e. as long as the structure is not able anymore of stand horizontal actions)

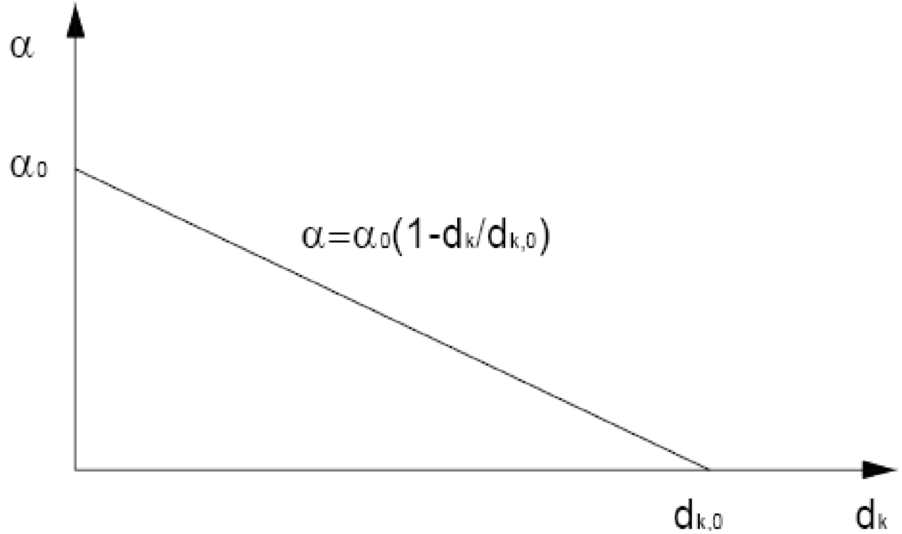
The α multiplier is introduced, ratio between the horizontal forces and a quantity depending of the corresponding weights of the present masses.



$$F_i = \alpha f\left(P_i\right)$$

The Linear Capacity Curve (I)

- The kinematic method allows to determine the linear capacity curve of the element under consideration



where:

α_0 is the coefficient that actives the mechanism

d_k is the displacement of the control point k (e.g. the centre of mass)

d_{k0} is the displacement of the control point k, in which the multiplication factor of the horizontal forces is equal to zero (α=0)

The Linear Capacity Curve (II)

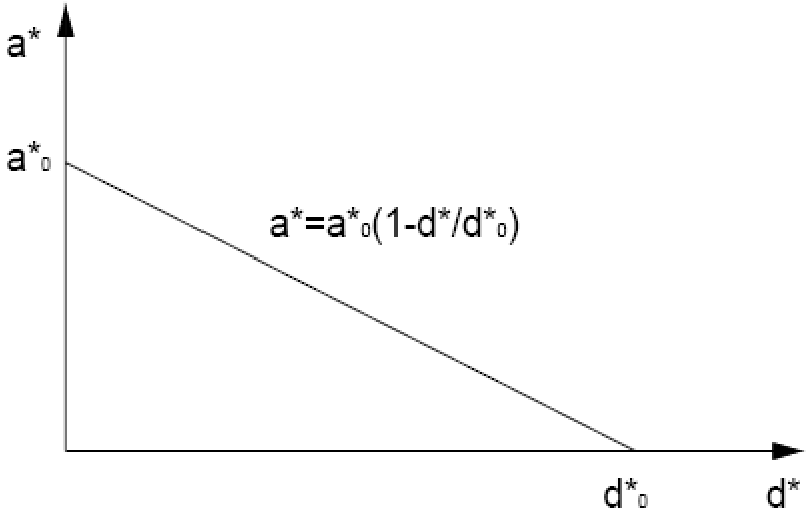
- The capacity curve is converted in the capacity curve of an equivalent SDOF system using the equations:

Spectral acceleration activating the mechanism

$$a_0^* = \frac{\alpha_0 \sum_{i=1}^{n+m} P_i}{M^*} = \frac{\alpha_0 g}{e^*}$$

Displacement of the equivalent system

$$d^* = d_k \frac{\sum_{i=1}^{n+m} P_i \delta_{x,i}}{\delta_{x,k} \sum_{i=1}^{n+m} P_i}$$



where:

Participating mass

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \delta_{x,i}\right)^2}{g \sum_{i=1}^{n+m} P_i \delta_{x,i}^2}$$

Fraction of the participating mass

$$e^* = g M^* / \sum_{i=1}^{n+m} P_i$$

The Linear Capacity Curve (III)

- The hypotheses of “hinging” limit analysis are usually considered:
  - Masonry withstand no tensile stresses
  - No sliding present in the mechanism
  - Compressive strength is infinite

- But it is possible to consider more realistic hypothesis:
  - Friction sliding
  - Connections, even if weak, between transverse and longitudinal walls
  - Ties
  - Limited compressive strength, with an internal position of the hinge
  - Walls with weak connection between leaves

The Linear Capacity Curve (IV)

- The α_0 coefficient is obtained from the Principle of Virtual Work (PVW)

- Applying a virtual rotation, the system is in equilibrium if the work due to external forces is equal to the work due to internal forces:

$$W_{ext} = W_{int}$$

$$\alpha_0 \left(\sum_{i=1}^{n} P_i \delta_{x,i} + \sum_{j=n+1}^{n+m} P_j \delta_{x,j} \right) - \sum_{i=1}^{n} P_i \delta_{y,i} - \sum_{h=1}^{o} F_h \delta_h = L_{fi}$$

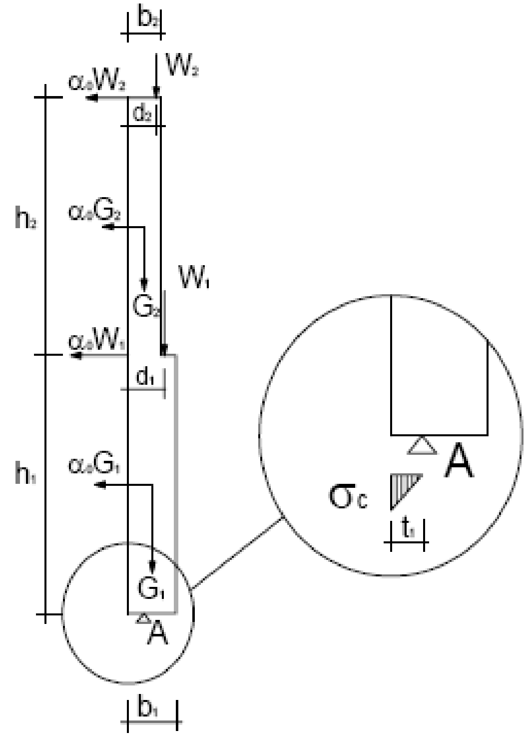
The Linear Capacity Curve (V)

- If only a rotation occurs, the PVW is equal to the balance of the vertical and horizontal forces acting around the hinging point

$$M_S = M_R$$

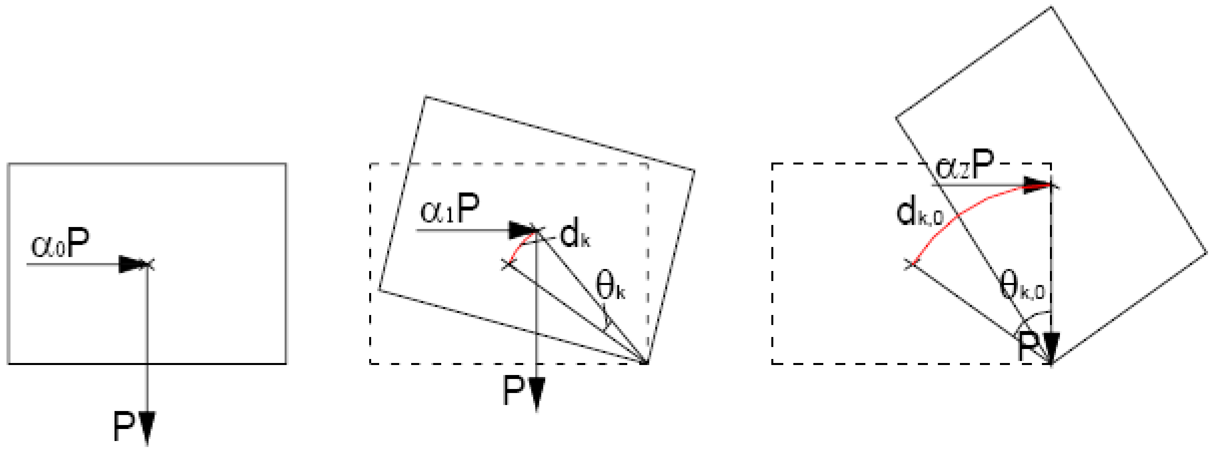
$$M_S = W_2(d_2 - t_1) + W_1(d_1 - t_1) + G_2\left(\frac{b_2}{2} - t_1\right) + G_1\left(\frac{b_1}{2} - t_1\right)$$

$$M_R = \alpha_0 W_2(h_1 + h_2) + \alpha_0 W_1 h_1 + \alpha_0 G_2\left(\frac{h_2}{2} + h_1\right) + \alpha_0 G_1 \frac{h_1}{2}$$



The Linear Capacity Curve (VI)

- Displacement $d_{k,0}$ is determined with the evolution of the mechanism, which in fact provides a non-linear relation



$\alpha_2=0$

Safety Verifications (OPCM 3274 / OPCM 3431)

DAMAGE LIMIT STATE: **The safety verification with reference to the DLS is satisfied when the spectral acceleration for the activation of the mechanism a^*_0 is greater than the acceleration of the elastic spectrum, evaluated for T=0, opportunely amplified in order to consider the portion of the building interested by the kinematic mechanism**

$$a_0^* \geq \frac{a_g S}{2.5}\left(1+1.5\frac{Z}{H}\right)$$

where:

Z **is the height of the center of the masses that generate horizontal forces on the elements of the kinematic chain, because they are not effectively transmitted to other parts of the buildings**

H **is the height of the whole structure**

Safety Verifications (OPCM 3274 / OPCM 3431)

- **In case of local mechanisms, the damage limit state corresponds to the arising of cracking that interests not the whole but only a part of the structure. Therefore, in case of existing masonry buildings, even if the fulfillment of this limit is desirable, its verification is not required.**

ULTIMATE LIMIT STATE: **The ultimate limit state verification of the local mechanism is instead MANDATORY, in order to assure the safety with respect of the collapse. This verification can be developed through the following criteria:**

- **Simplified verification with structure factor q (linear kinematic analysis)**
- **Verification through capacity spectrum (non-linear kinematic analysis)**

Safety Verifications (OPCM 3274 / OPCM 3431)

Simplified verification with structure factor q (linear kinematic analysis)

The verification is satisfied if:

$$a_0^* \geq \frac{a_g S}{q}\left(1+1.5\frac{Z}{H}\right)$$

where q is the structure factor assumed equivalent to 2

Verification through capacity spectrum (non-linear kinematic analysis)

First of all it is necessary to define the spectral displacement d^*_u

$$d^*_u = \min \begin{cases} 0.4 d^*_0 \\ \text{Displacement corresponding to situations locally not compatible with the stability of the construction elements (beam slipping, etc…)} \end{cases}$$

Safety Verifications (OPCM 3274 / OPCM 3431)

Verification through capacity spectrum (non-linear kinematic analysis)

The verification is satisfied if:

$$\Delta_d \leq d_u^*$$

where:

d^*_u represents the ultimate displacement capacity of the system

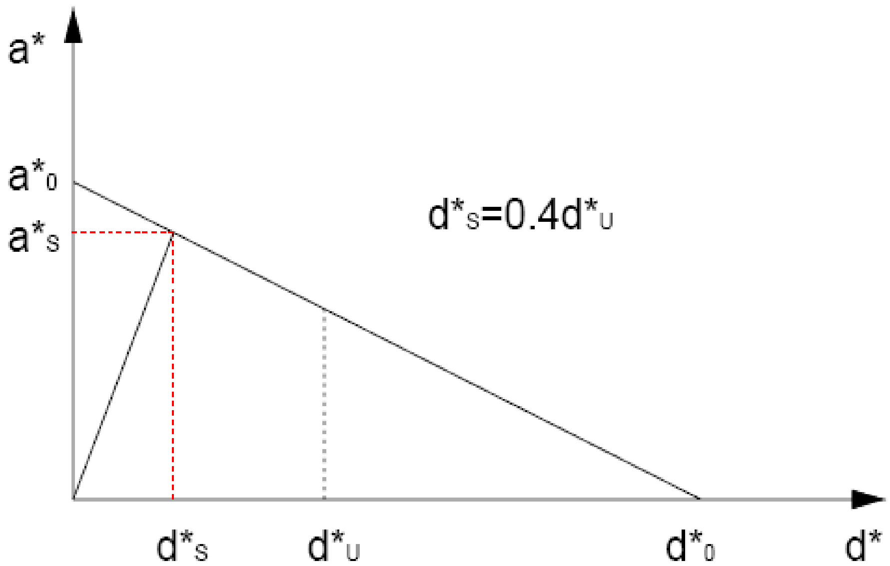
Δ_d is the displacement demand of the earthquake, evaluated through the spectrum defined similarly to the one utilized for the verification of the non structural elements in correspondence to the secant period T_s:

$$T_s = 2\pi\sqrt{\frac{d_s^*}{a_s^*}}$$

Safety Verifications (OPCM 3274 / OPCM 3431)

Verification through capacity spectrum (non-linear kinematic analysis)

Parameters a^*_s and d^*_s are identified on the capacity curve in the following way



Safety Verifications (OPCM 3274 / OPCM 3431)

Verification through capacity spectrum (non-linear kinematic analysis)

The displacement $\Delta_d(T_s)$ required by the earthquake is determined as:

$$T_s < 1.5T_1 \qquad \Delta_d(T_s) = a_g S \frac{T_s^2}{4\pi^2}\left(\frac{3(1+Z/H)}{1+(1-T_s/T_1)^2} - 0.5\right)$$

$$1.5T_1 \leq T_s < T_D \qquad \Delta_d(T_s) = a_g S \frac{1.5T_1 T_s}{4\pi^2}\left(1.9 + 2.4\frac{Z}{H}\right)$$

$$T_D \leq T_s \qquad \Delta_d(T_s) = a_g S \frac{1.5T_1 T_D}{4\pi^2}\left(1.9 + 2.4\frac{Z}{H}\right)$$

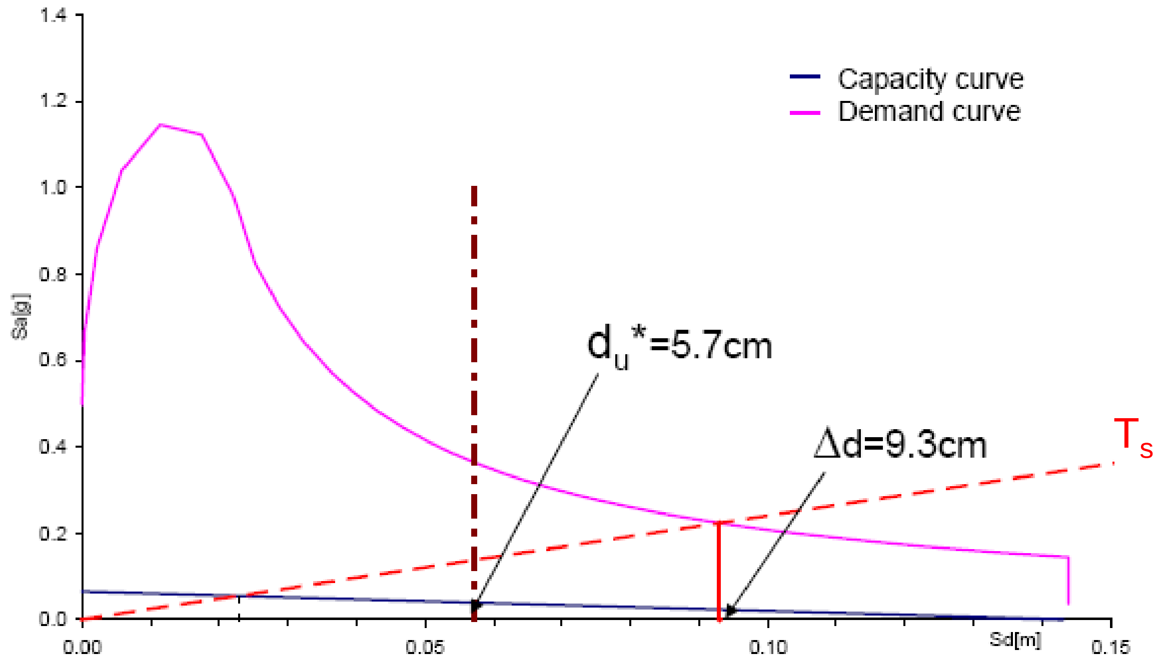
where:

T_1 is the first period of the structure in the direction being considered

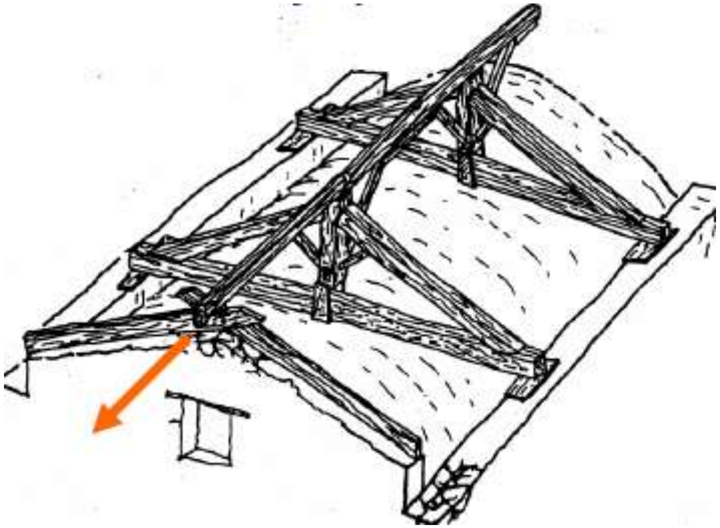
Safety Verifications (OPCM 3274 / OPCM 3431)

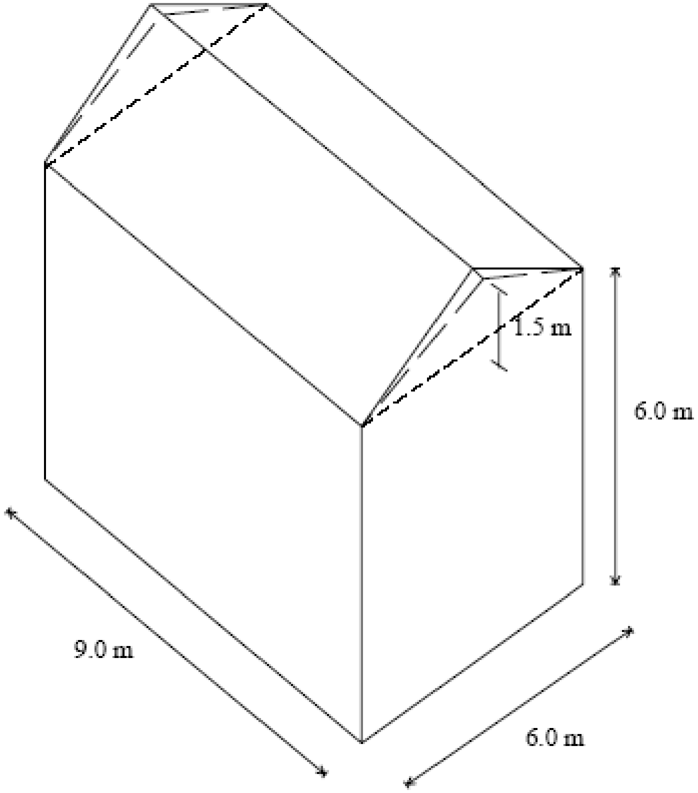
Verification through capacity spectrum (non-linear kinematic analysis)

Graphic interpretation of the ADSR diagram (acceleration-displacement)

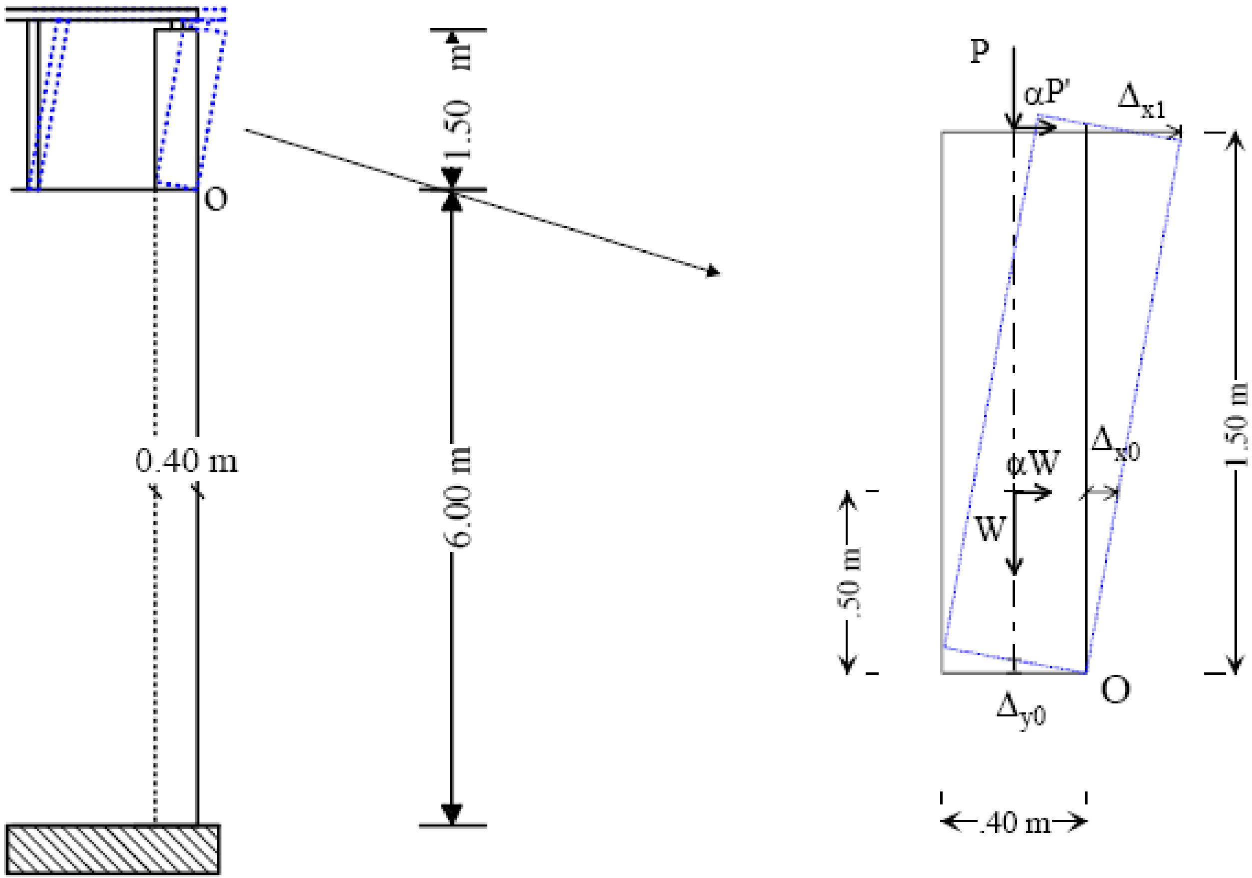


Examples #1: Overturning of a frontispiece

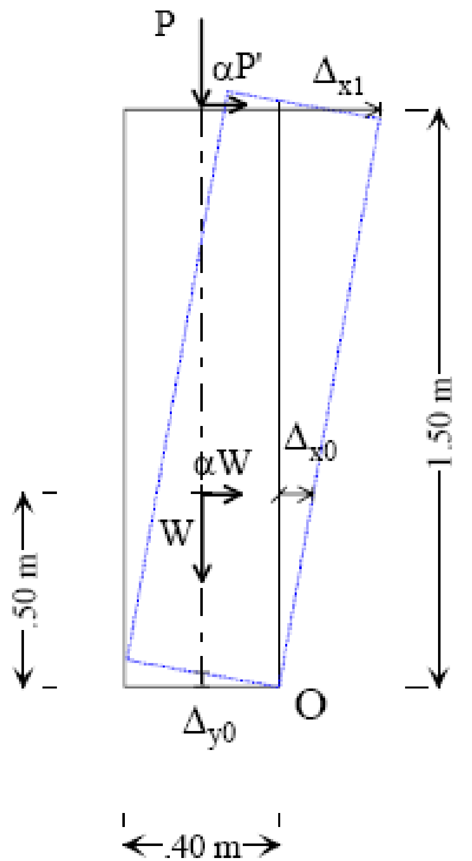




Examples #1: Overturning of a frontispiece



Examples #1: Overturning of a frontispiece



Actions

Deadweight from roof: 1.4 KN/m^2

Specify weight for masonry: 20 KN/m^3

Then, P=26 KN

P'=1/2 of the weight over the ridge, P' = 39 KN

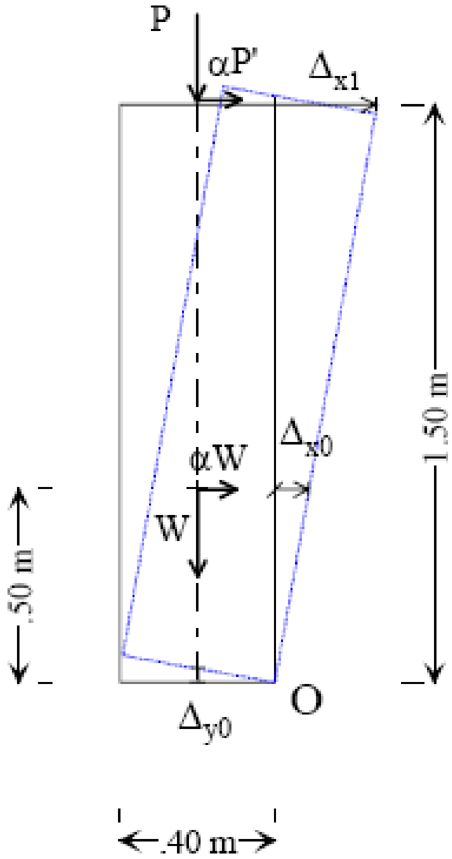
Position of the wall weight: 1/3 of the height

Calculation of α_0 (equilibrium)

$$\alpha_0(1.5 \cdot P' + 0.5 \cdot W) - 0.2 \cdot P - 0.2 \cdot W = 0$$

$$\alpha_0 = \frac{0.2 \cdot P + 0.2 \cdot W}{1.5 \cdot P' + 0.5 \cdot W} = 0.162$$

Examples #1: Overturning of a frontispiece



Calculation of α_0 using the PVW

$$\alpha_0 P' \cdot \Delta_{x1} + \alpha_0 W \cdot \Delta_{x0} - P \cdot \Delta_{y0} - W \cdot \Delta_{y0} = 0$$

$$\Delta_{x0} = \theta \cdot 0.5; \quad \Delta_{x1} = \theta \cdot 1.5; \quad \Delta_{y0} = \theta \cdot 0.2$$

$$\alpha_0 P' \cdot 1.5\theta + \alpha_0 W \cdot 0.5\theta - P \cdot 0.2\theta - W \cdot 0.2\theta = 0$$

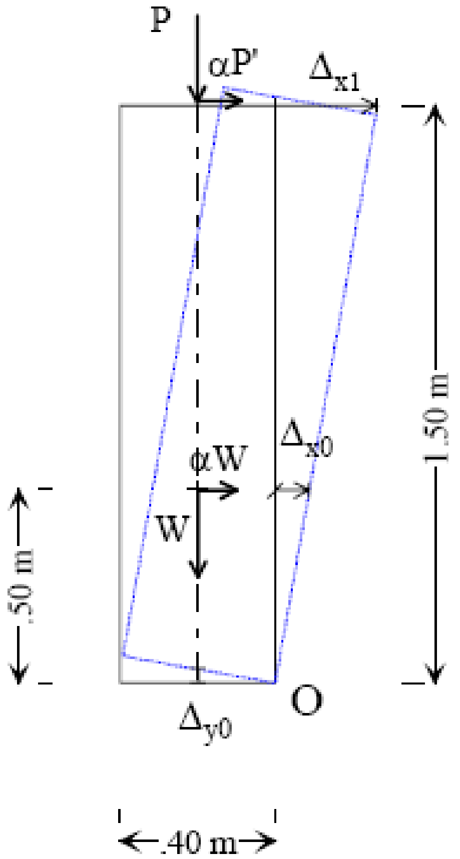
$$\alpha_0 = \frac{0.2 \cdot P + 0.2 \cdot W}{1.5 \cdot P' + 0.5 \cdot W} = 0.162$$

Participating mass M*

Using D_{x1} as control displacement, $D_{X0}=D_{x1}/3$

$$M^* = \frac{\left(\sum_{i=1}^{n+m} P_i \delta_{x,i}\right)^2}{g \sum_{i=1}^{n+m} P_i \delta_{x,i}^2} = \frac{(P' \cdot 1 + W \cdot 0.333)^2}{9.81 \cdot (P' \cdot 1^2 + W \cdot 0.333^2)} = 6.162$$

Examples #1: Overturning of a frontispiece



Fraction of participating mass e*

$$e^* = gM^* / \sum_{i=1}^{n+m} P_i = \frac{9.81 \cdot 6.162}{W + P'} = 0.806$$

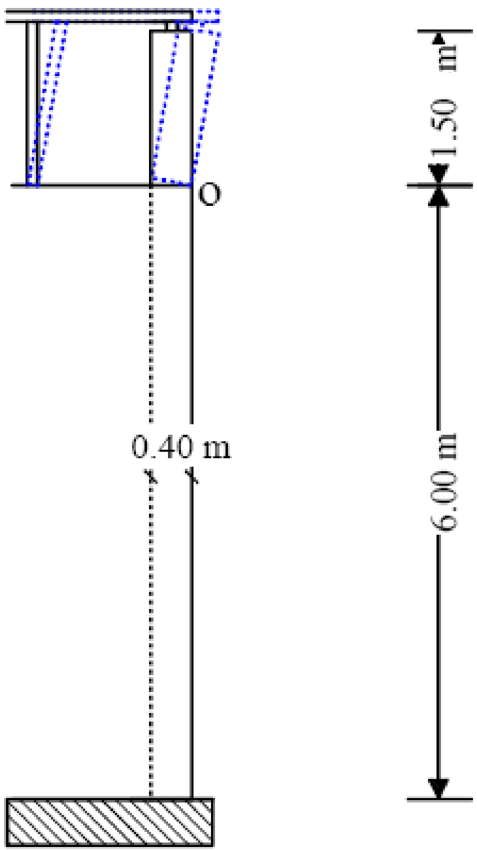
Spectral acceleration for mechanism activation a^*_0

$$a_0^* = \frac{\alpha_0 \sum_{i=1}^{n+m} P_i}{M^*} = \frac{\alpha_0 g}{e^*} = \frac{0.162 \cdot 9.81}{0.806} = 1.972 \text{ m/s}^2$$

Safety verification using linear analysis (ULS)

$$a_0^* \geq \frac{a_g S}{q}\left(1 + 1.5\frac{Z}{H}\right)$$

Examples #1: Overturning of a frontispiece



Safety verification using linear analysis (ULS)

$$a_0^* \geq \frac{a_g S}{q}\left(1 + 1.5\frac{Z}{H}\right)$$ with q = 2.0

$$\frac{Z}{H} = 0.936$$ where: $Z = 7,02 = (W.h_w + P'.h_{P'})/(W+P')$

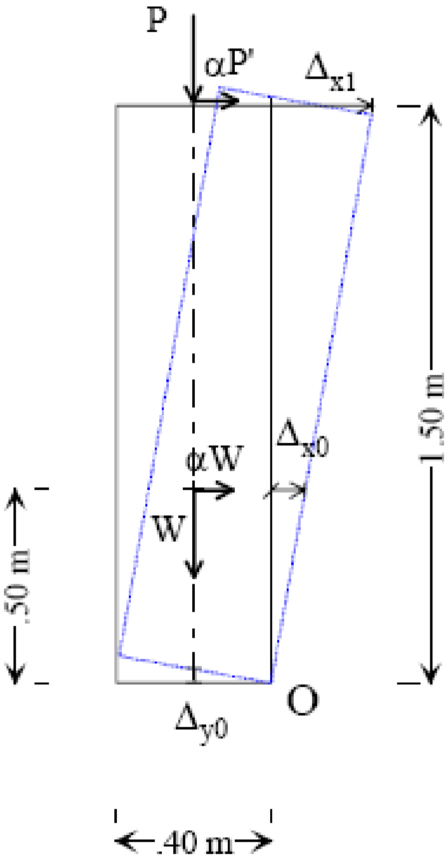
$H = 7.5$ (total building height)

$$a_0^* \geq \frac{a_g S}{2.0}(1 + 1.5 \cdot 0.936) = 1.202 a_g S$$

Safety is verified if:

$$a_g S \leq a_0^*/1.202 = 1.641 \text{ m/s}^2 = 0.167g$$

Examples #1: Overturning of a frontispiece



Verification using the capacity spectrum (ULS)

Static relation a-D_{x1} (a-d_k) is determined for displacements different from zero. All forces are proportional to the weight, and the relation a-d_k can be assumed linear:

$$\alpha = \alpha_0 \left(1 - d_k / d_{k,0}\right)$$

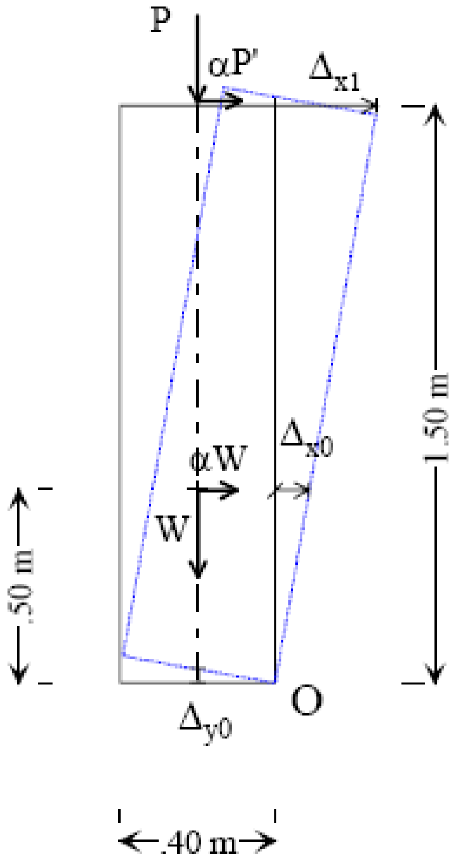
Displacement d_{k0} is calculated for α=0

$$W \cdot (0.2 - \Delta_{x0}) - P \cdot (0.2 - \Delta_{x1}) = 0$$

$$W \cdot (0.2 - \Delta_{x1}/3) - P \cdot (0.2 - \Delta_{x1}) = 0$$

$$\Delta_{x1,0} = d_{k,0} = \frac{W \cdot 0.2 + P \cdot 0.2}{W/3 + P} = 0.326\ \text{m}$$

Examples #1: Overturning of a frontispiece



Equivalent 1 dof system displacement

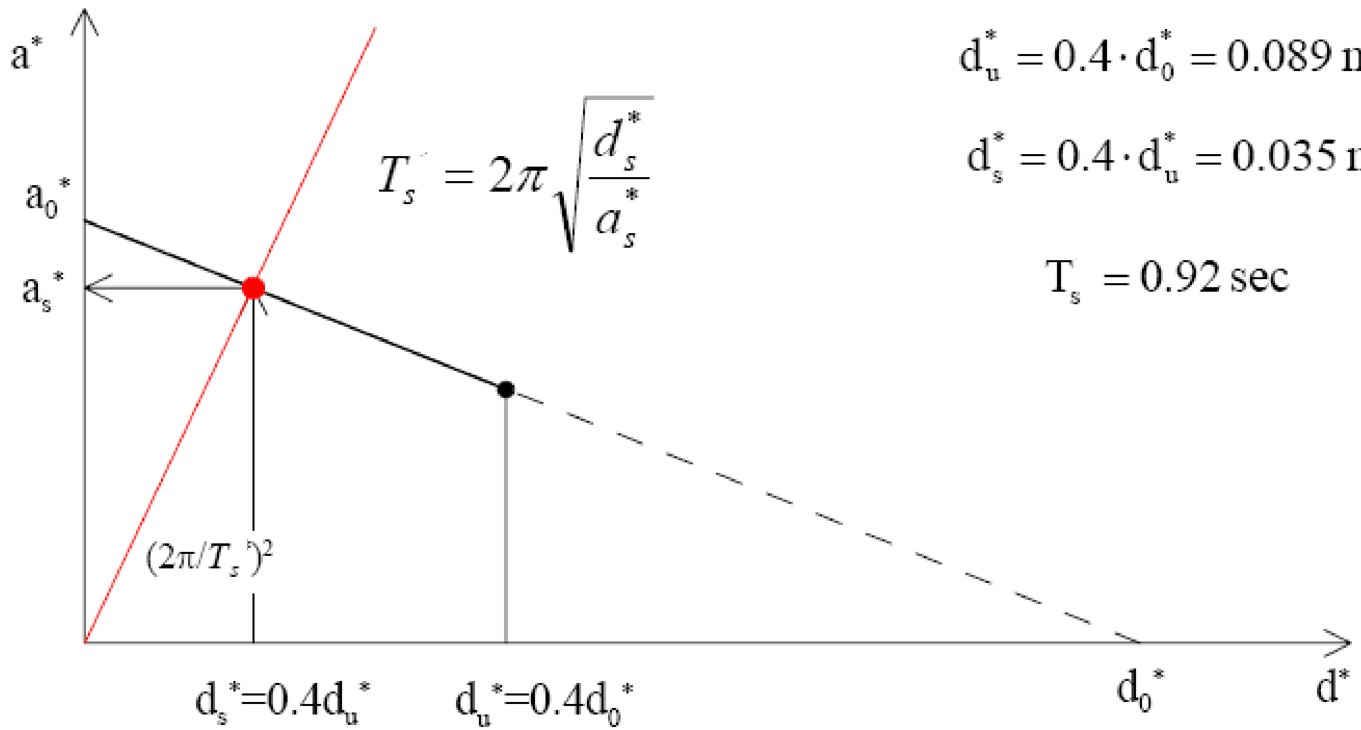
$$d^* = d_k \frac{\sum_{i=1}^{n+m} P_i \delta_{x,i}}{\delta_{x,k} \sum_{i=1}^{n+m} P_i} = d_k \frac{P' \cdot 1 + W \cdot 0.333}{1 \cdot (P' + W)} = 0.68 \cdot d_k$$

$$d_0^* = 0.68 \cdot d_{k,0} = 0.222\,\mathrm{m}$$

$$d_u^* = 0.4 \cdot d_0^* = 0.089\,\mathrm{m}$$

$$d_s^* = 0.4 \cdot d_u^* = 0.035\,\mathrm{m}$$

Examples #1: Overturning of a frontispiece



$$d_0^* = 0.68 \cdot d_{k,0} = 0.222\ \text{m}$$

$$d_u^* = 0.4 \cdot d_0^* = 0.089\ \text{m}$$

$$d_s^* = 0.4 \cdot d_u^* = 0.035\ \text{m}$$

$$T_s = 0.92\ \text{sec}$$

Examples #1: Overturning of a frontispiece

Assuming a fundamental period for the building equal to 0.2 s (T_1 = 0.2 s):

$$T_s = 0.92 > 1.5\ T_1 = 0.3\text{s}$$

Also assuming $a_g S$ = 0.2g = 1.962 m/s^2, the demand in displacement is given by:

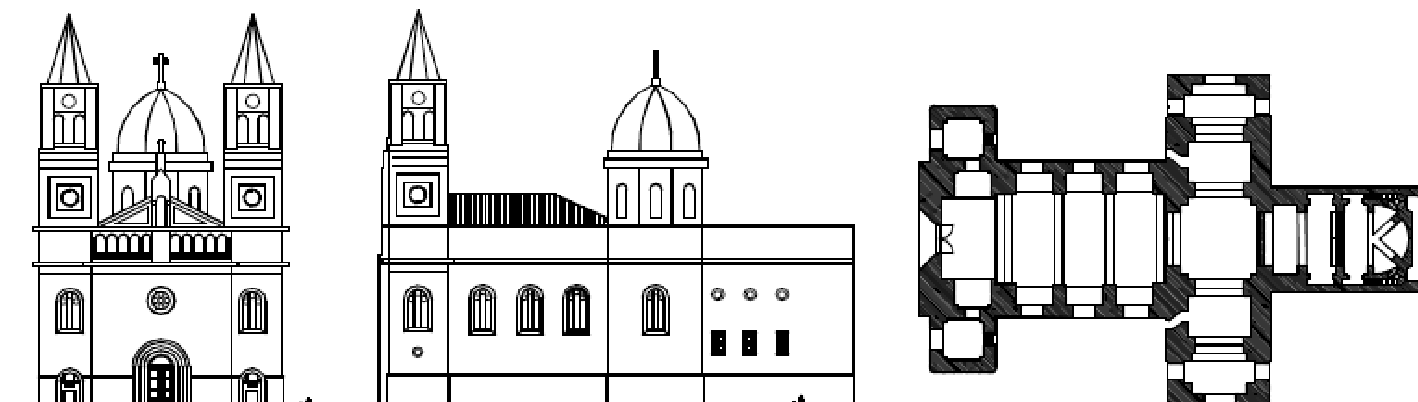
$$\Delta(T_s) = a_g S \cdot 1.5 \cdot \frac{T_1 T_s}{4\pi^2}\left(1.9 + 2.4\frac{Z}{H}\right) = 0.071\,\text{m} < 0.089\,\text{m} = \text{d}_\text{u}^*$$

Examples #2: Full Church

Location: Guimarães

Style: hybrid with classic, gothic, renaissance and romantic elements

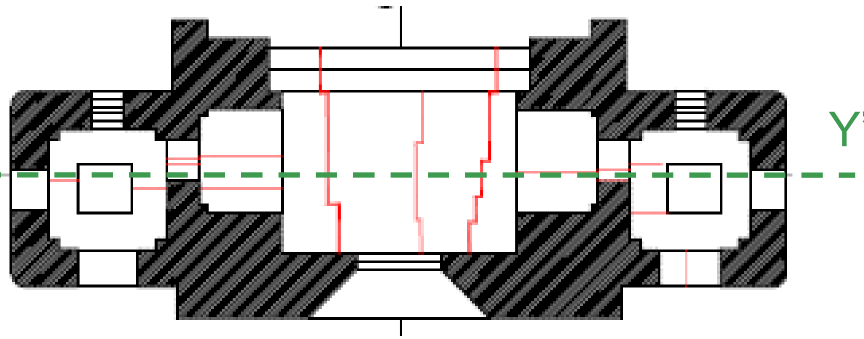
Material: granite ashlar masonry



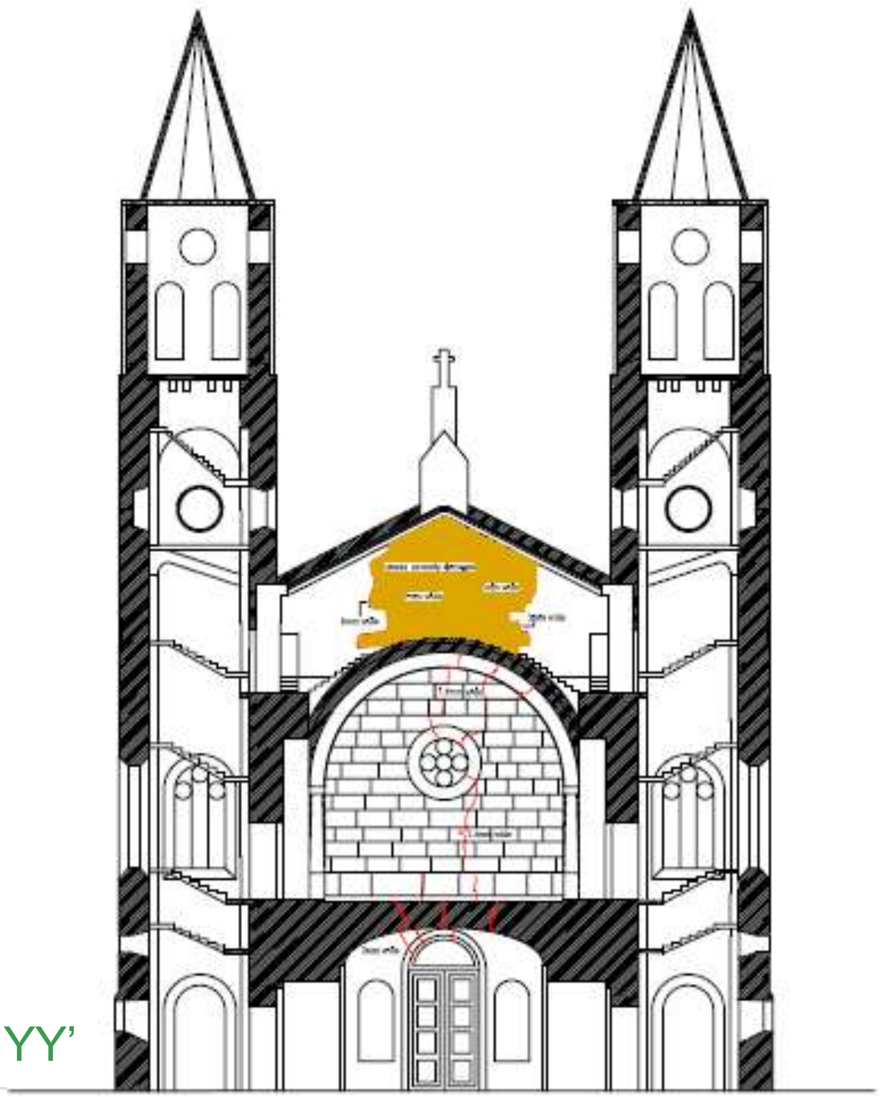
Examples #2: Full Church

- Cracking Pattern

Y



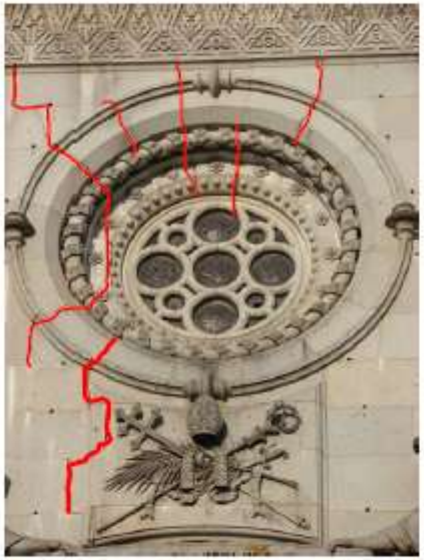
Y'

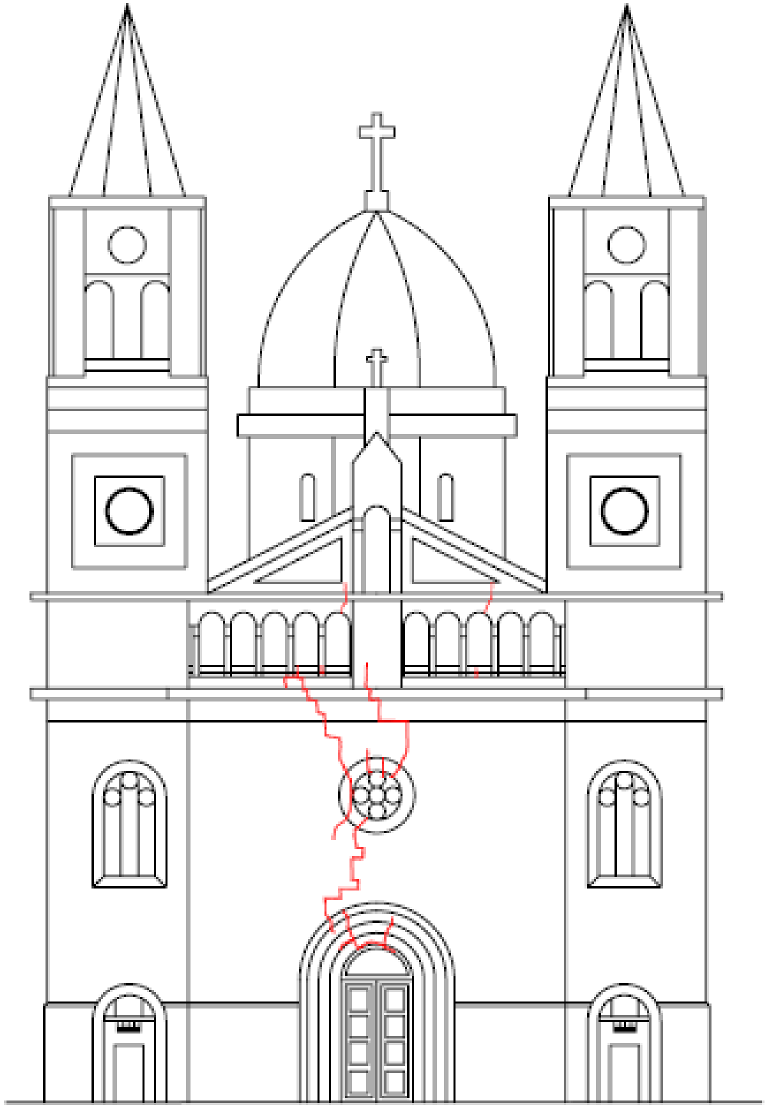


Corte YY'

Examples #2: Full Church

- Cracking pattern

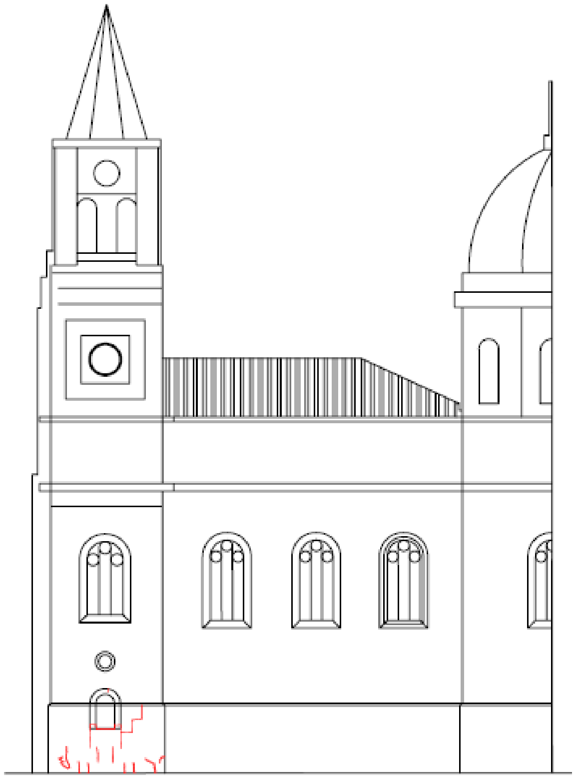




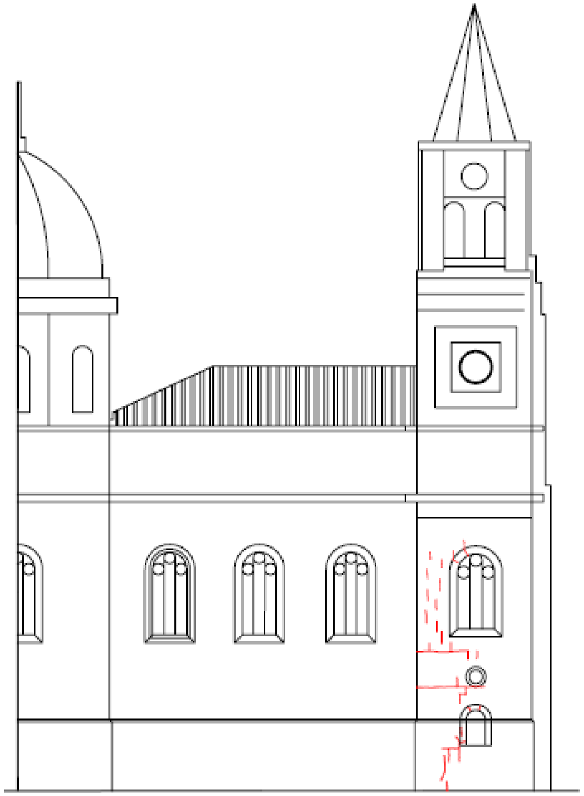
Main façade

Examples #2: Full Church

- Cracking pattern

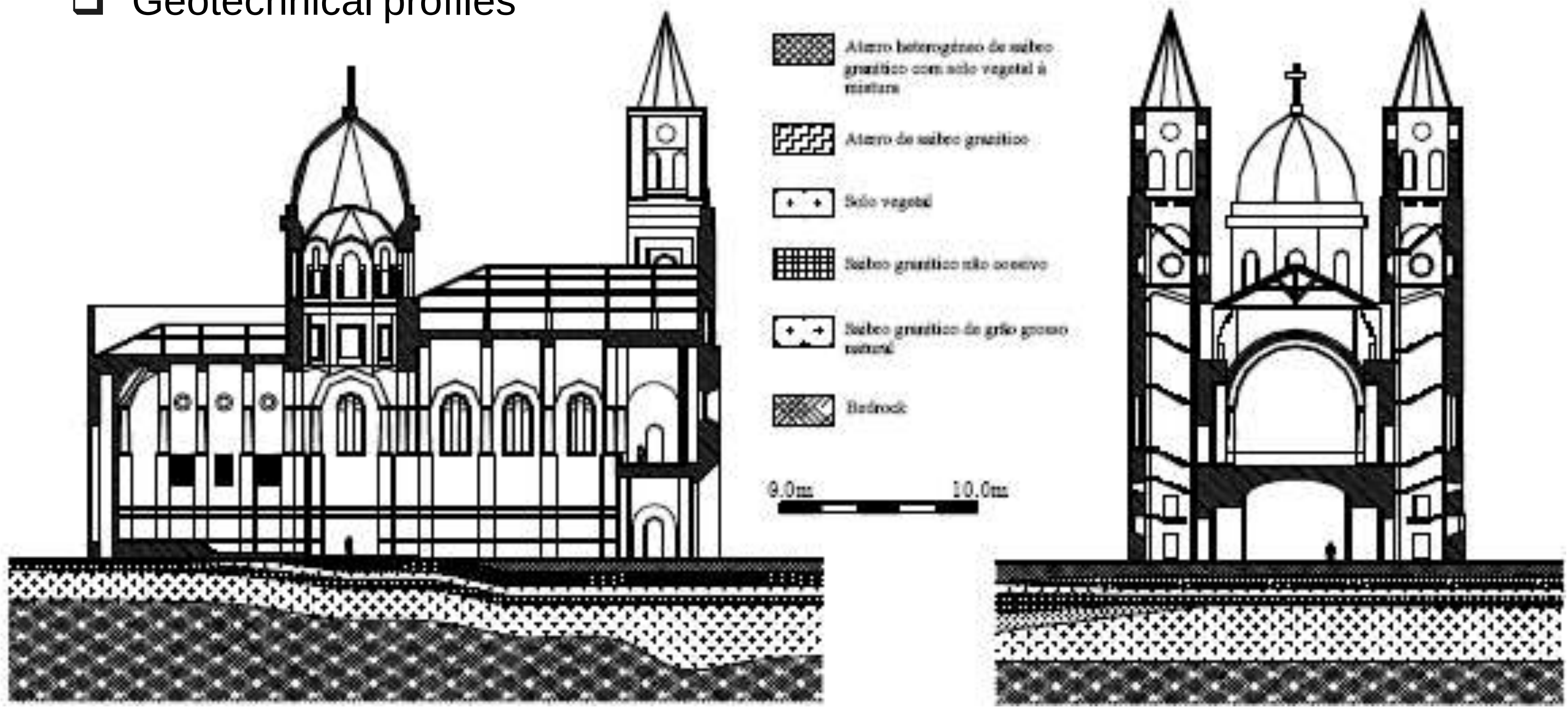


Views



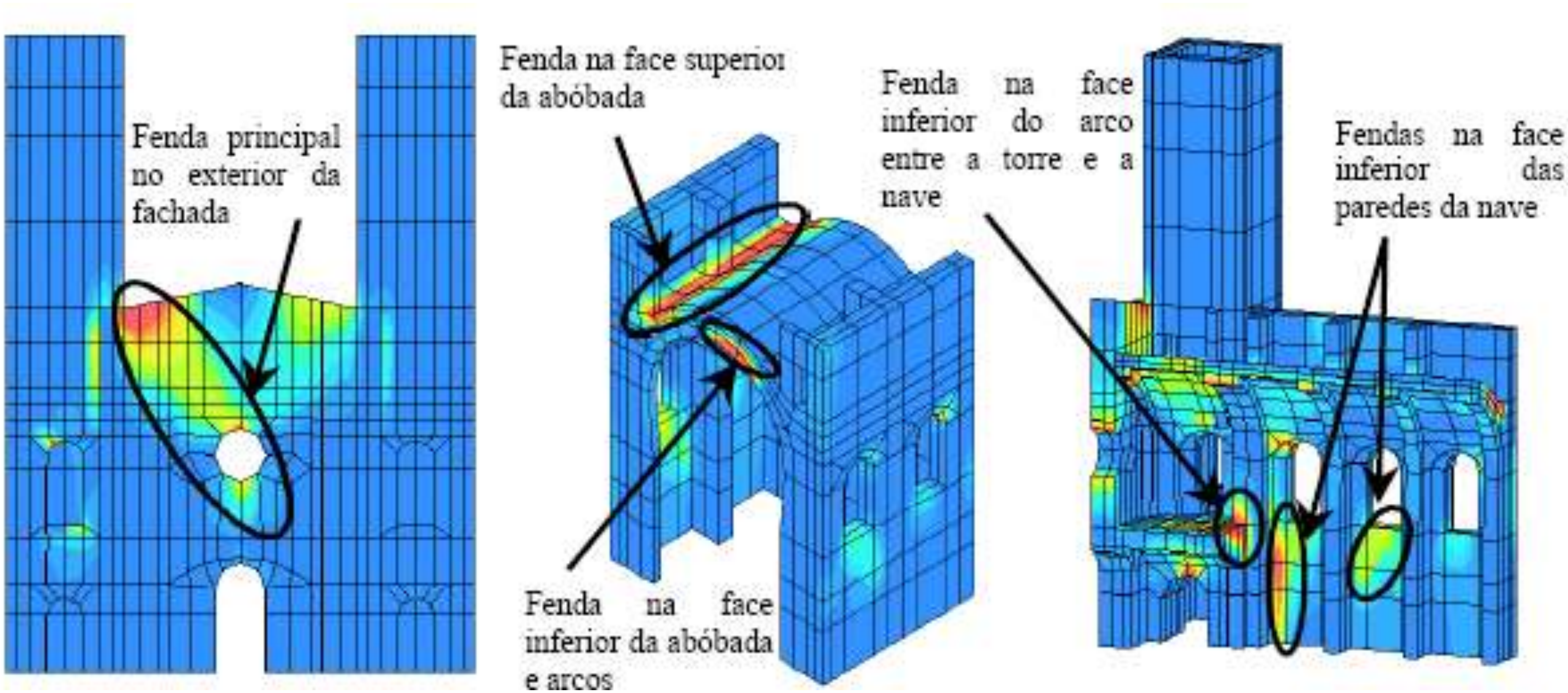
Examples #2: Full Church

- Geotechnical profiles



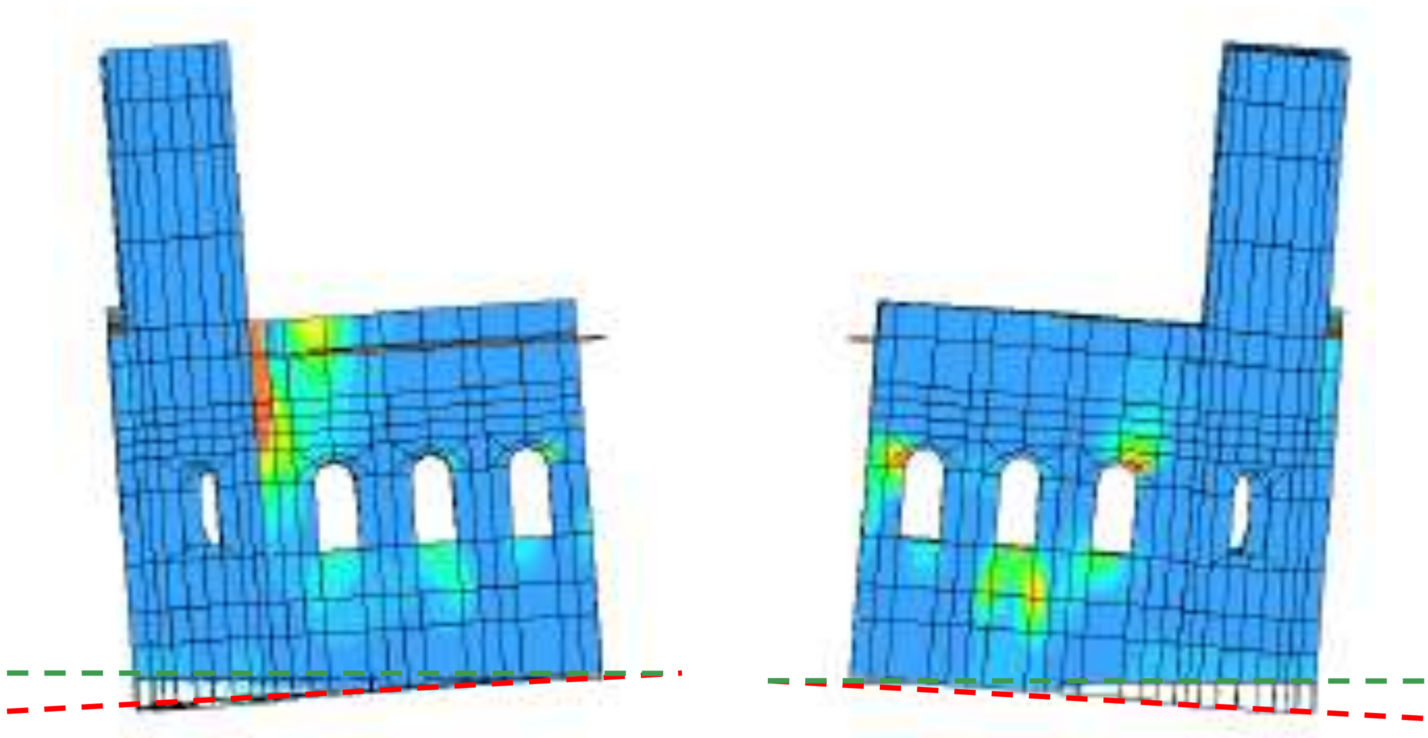
Examples #2: Full Church

- FEM Model



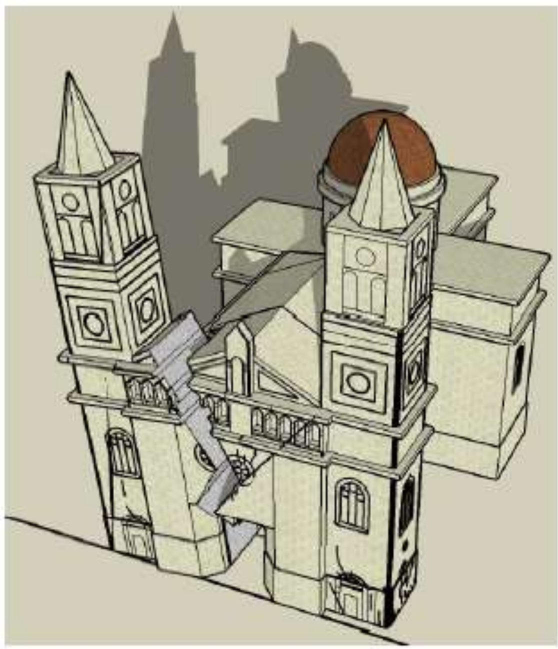
Examples #2: Full Churc

- Soil structure-**interaction**



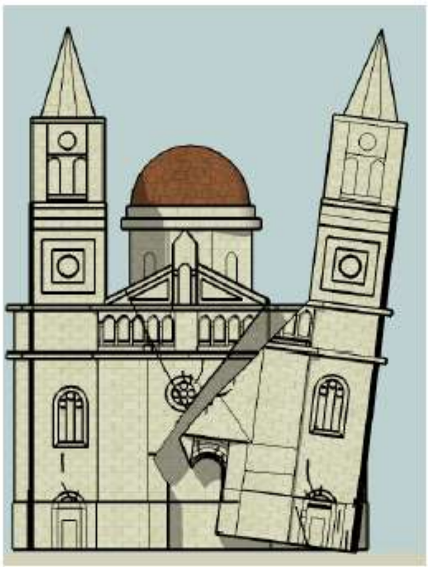
Examples #2: Full Church

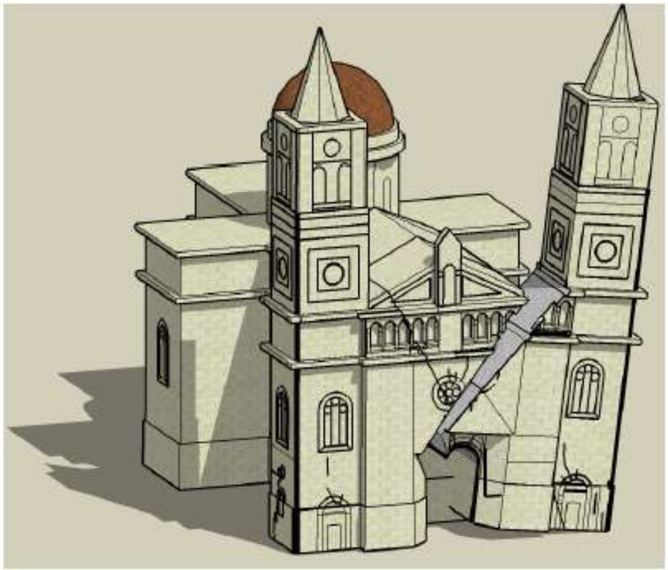
- Four possible mechanisms



	α_0	0.186
	M*	4343.7 KN
	e*	0.947 m/s^2
Capacity	a_0*	0.197 g
Demand	a_0*	0.063 g
Safety factor		3.13

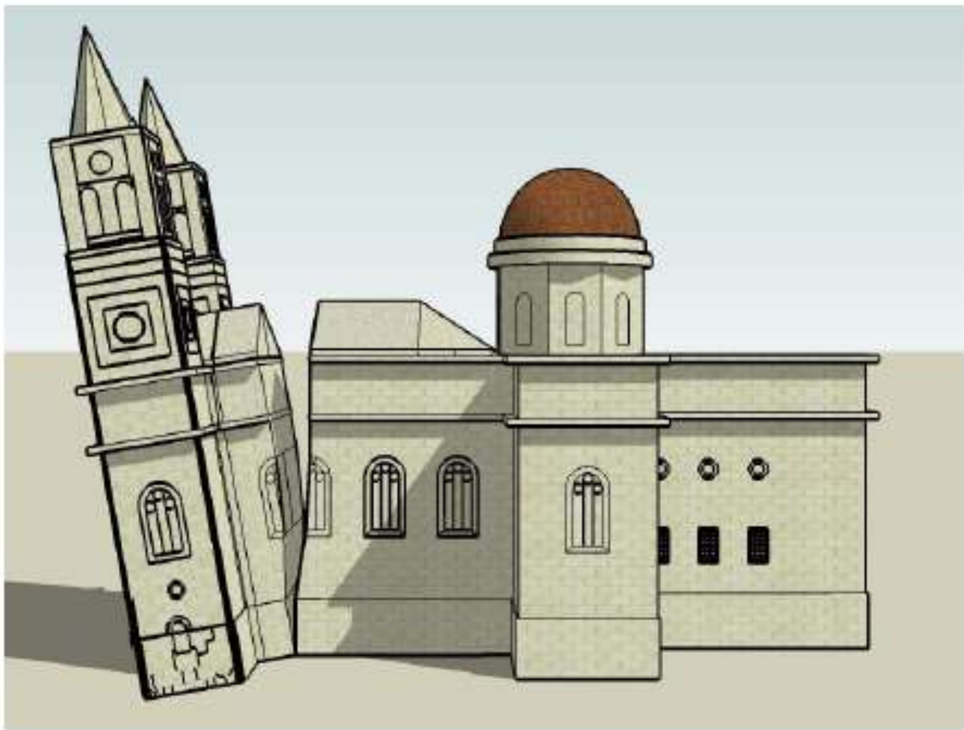
Examples #2: Full Church





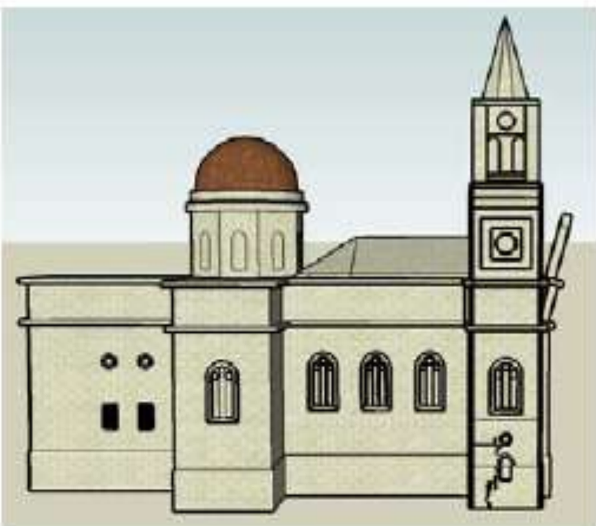
	α_0	0.184
	M*	4254.5 KN
	e*	0.953 m/s^2
Capacity	a_0*	0.193 g
Demand	a_0*	0.086 g
Safety Factor		2.24

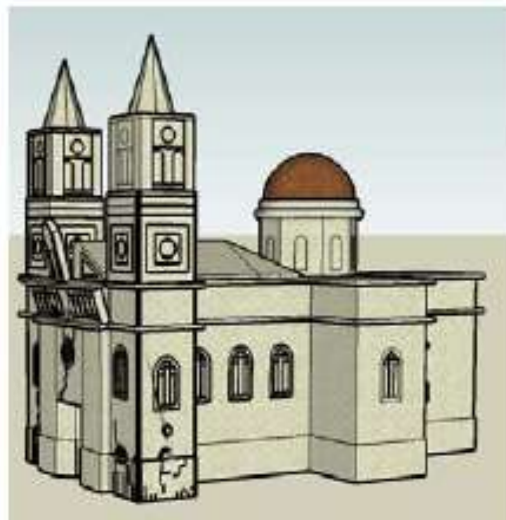
Examples #2: Full Church

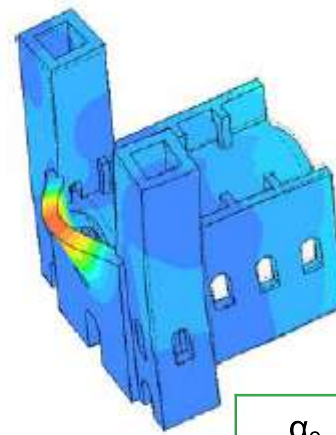


	α_0	0.164
	M*	8830.1 KN
	e*	0.968 m/s^2
Capacity	a_0*	0.169 g
Demand	a_0*	0.087 g
Safety Factor		1.94

Examples #2: Full Church



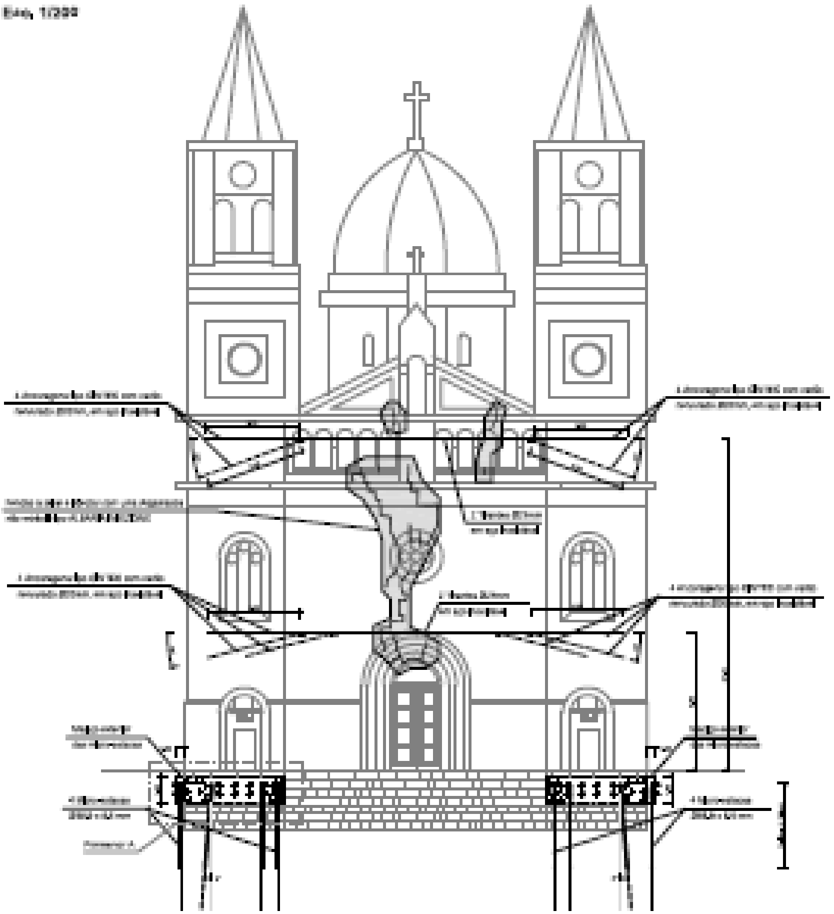




	α_0	0.205
	M*	339.1 KN
	e*	0.982 m/s^2
Capacity	a_0*	0.208 g
Demand	a_0*	0.123 g
Safety Factor		1.69

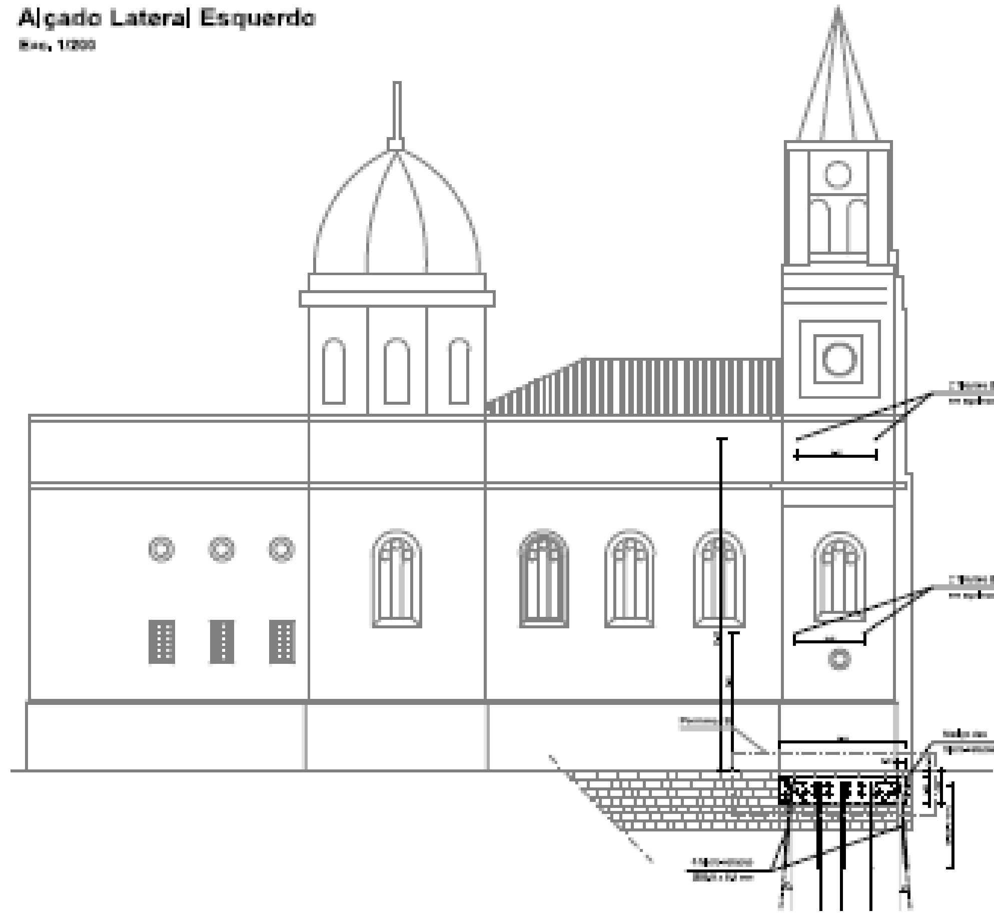
Alçado Principal

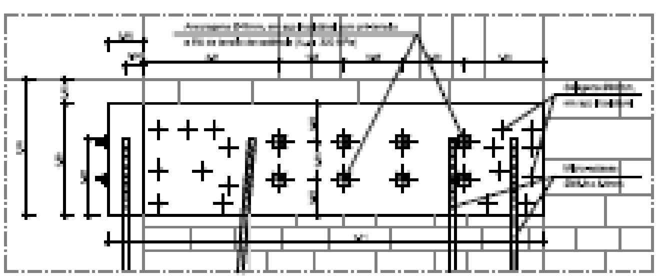
Esc. 1/200



Alçado Lateral Esquerdo

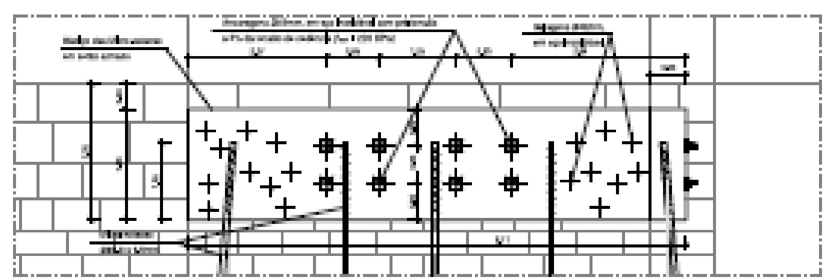
Esc. 1/200





Pormenor A

Esc. 1/50

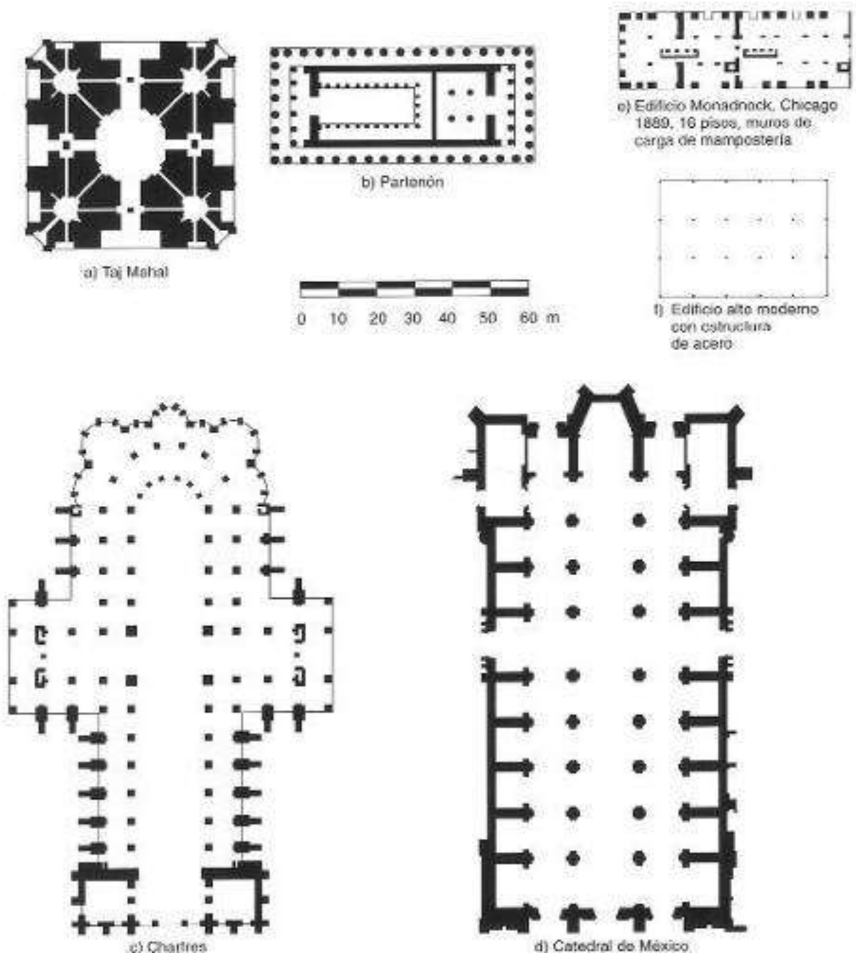


Pormenor B

Esc. 1/50

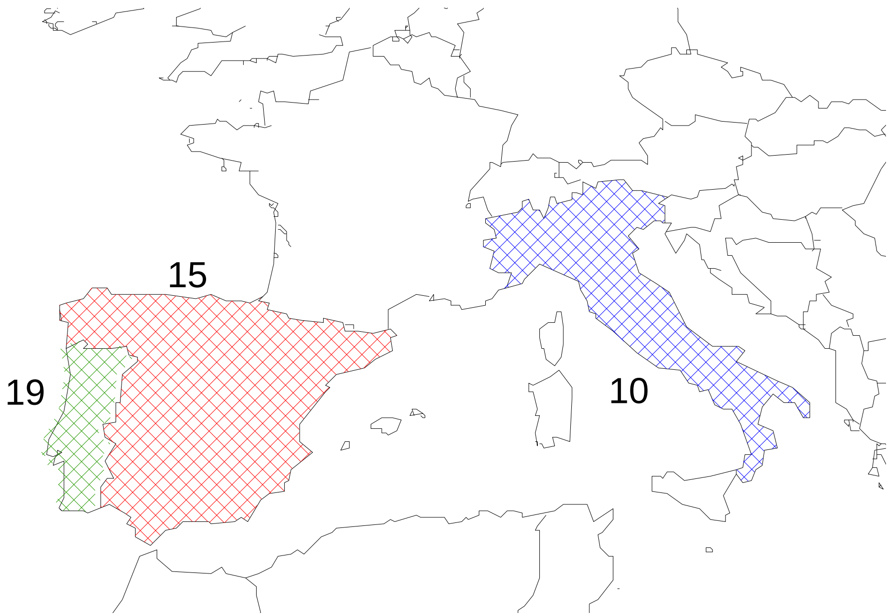
Simple Indicators for Seismic Performance

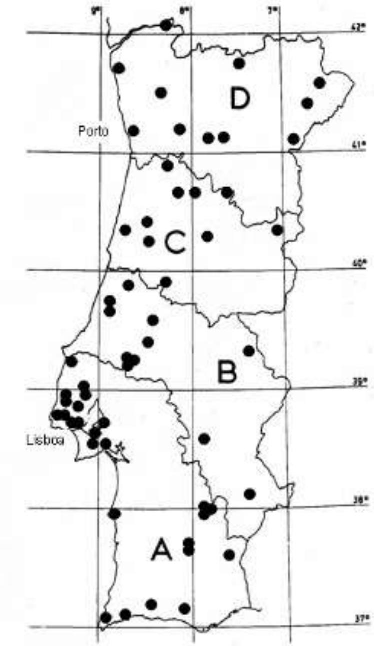
Idea: To have simple indicators to provide a first screen at territorial level

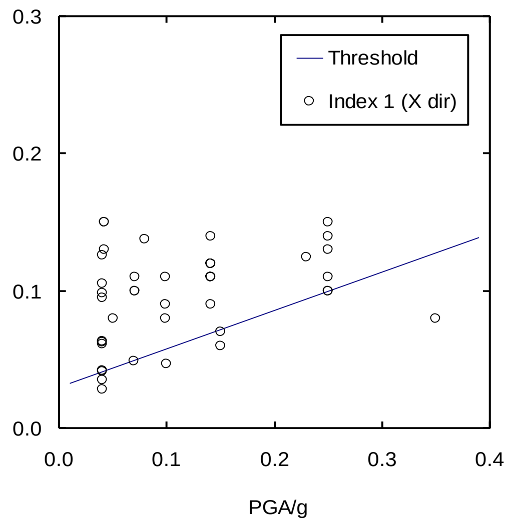


Different structural arrnagements

Idea: To have simple indicators to provide a first screen







European sample with 44 monuments

Portuguese sample with 55 monuments

Results

Index 1: In plan area

- This index provides a relation between the shear walls in each direction and the total area of the building. Only with a thickness larger than 0.35 m and with a ratio between the height and the width smaller than nine are considered as shear resistant

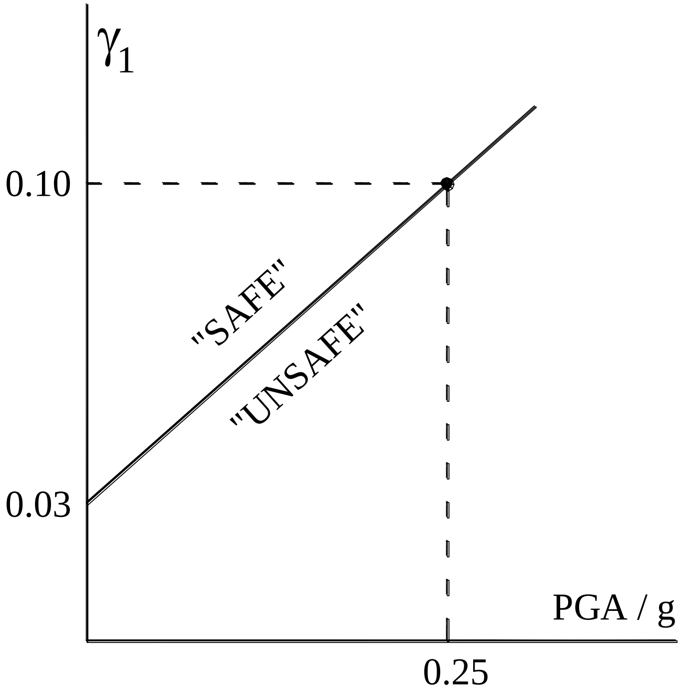
- The index is given by

$$\gamma_{1,i} = A_{wi} / S$$

where A_{wi} is the in plan are of the walls in direction "i" and S is the in plan area of the building

- The recommended value to trigger the need of more advanced studies is given by:

$$\gamma_{1,i} < 0.03 + 0.28\ PGA / g$$



Index 2: Ratio Between the Weight and the In Plan Area

- The index is given by

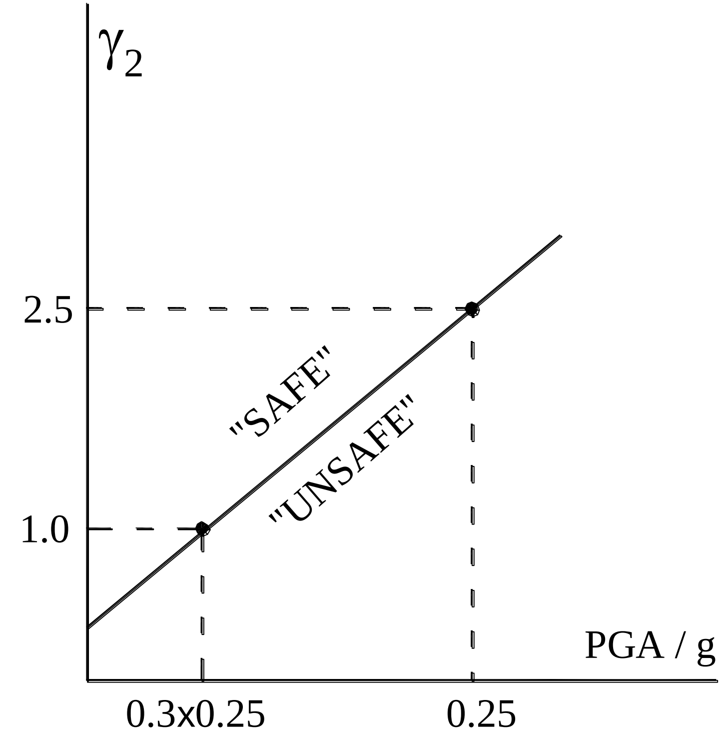
  $$\gamma_{2,i} = A_{wi} / G$$

  where G is the vertical quasi-permanent action

- This index measures the are per unit of weight, meaning that the building height (or mass) are considered. An important disadvantage is that the index has a dimension

- The recommended value to trigger the need of more advanced studies is given by:

  $$\gamma_{2,i} < 0.35 + 8.5\ PGA / g\ [m^2/MN]$$



Index 3: Base Shear Ratio

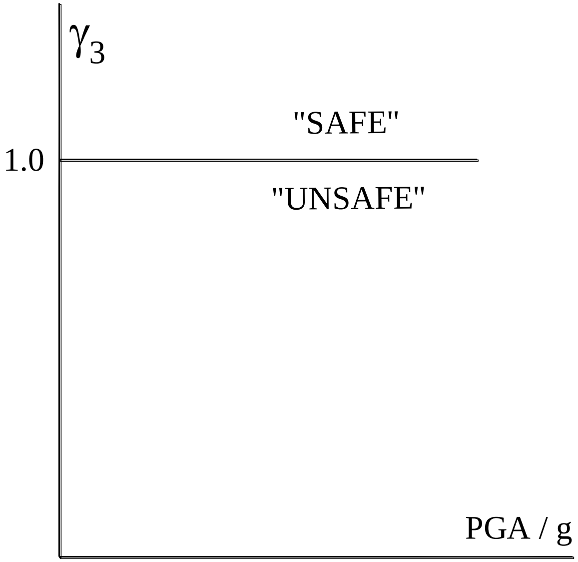
- The base shear V_{Sd} can be estimated using an equivalent "static" analysis applying horizontal forces given by (β.G), where β is a static coefficient related with the design acceleration

- The index is given by

$$\gamma_{3,i} = V_{Rd,i} / V_{Sd} = A_{wi} / A_w \times [\tan\phi + f_{vk0} / (\gamma \times h)] / \beta$$

where A_w is the total in plan area of the walls,
h is the average height of the building,
γ is the specific weight, ϕ is the friction angle (0.4) and f_{vk0} is the cohesion (0.05N/mm^2)

- The recommended value to trigger the need of more advanced studies is given by:
$\gamma_{3,i} < 1.0$

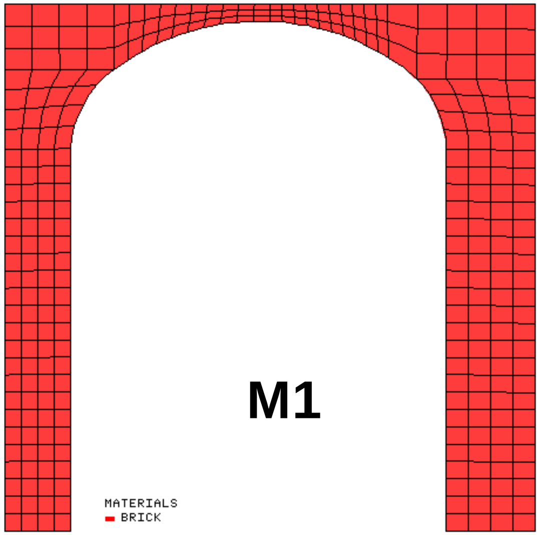


isise

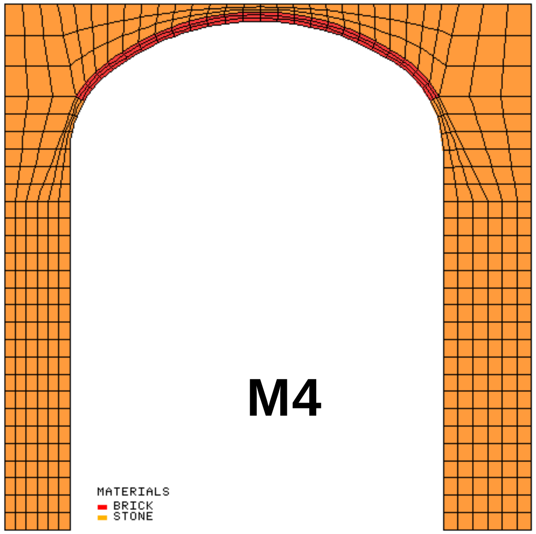
Examples and Reflections

Universidade do Minho

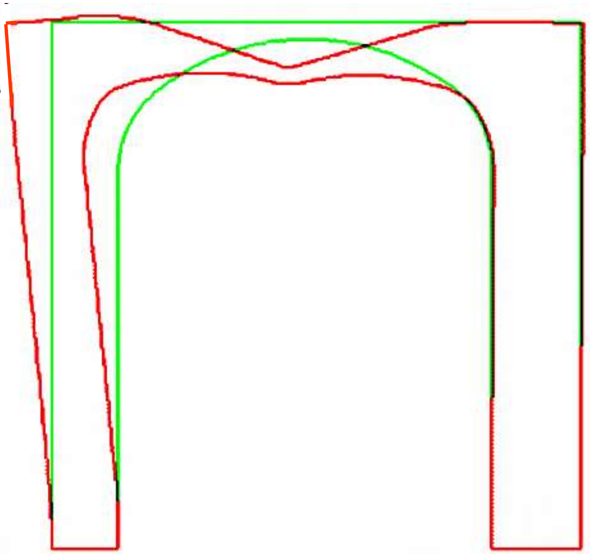
Lethes Theatre, Faro



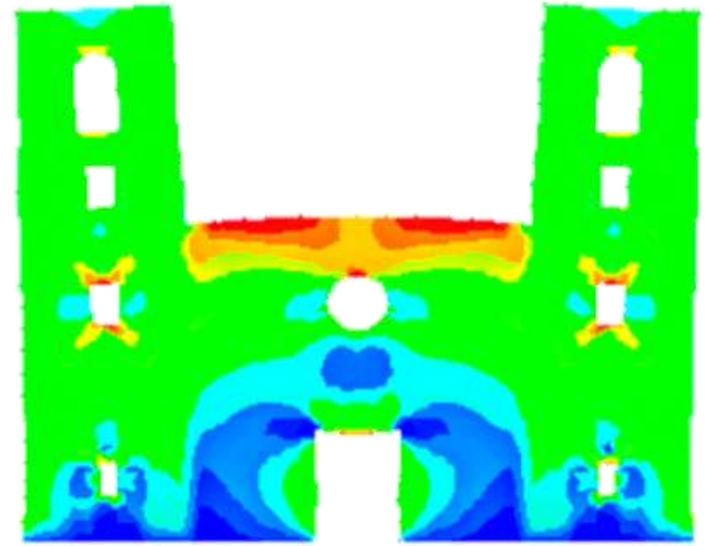
M1

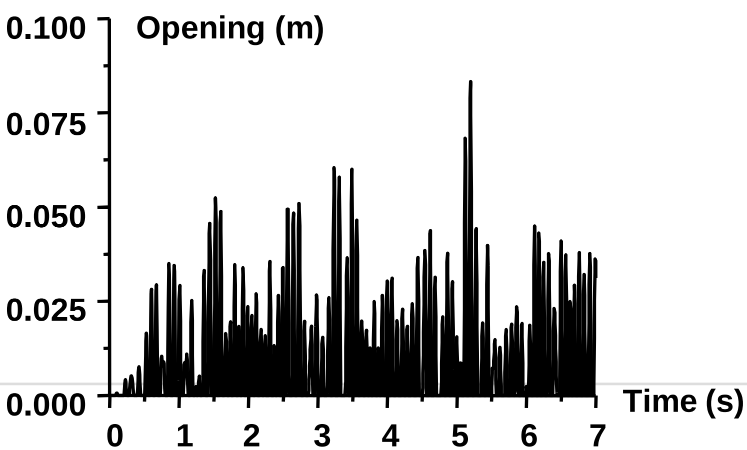


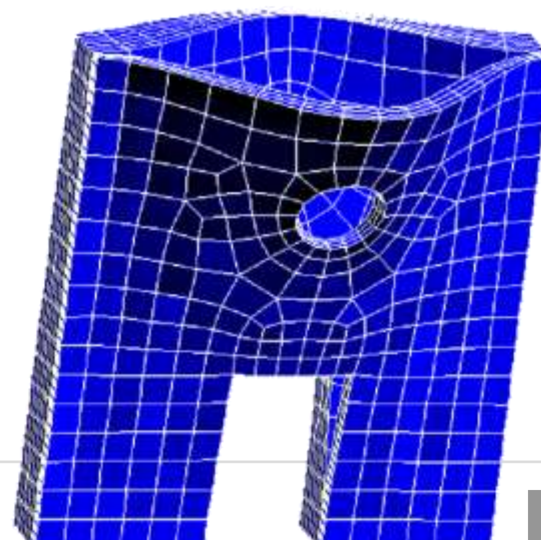
M4



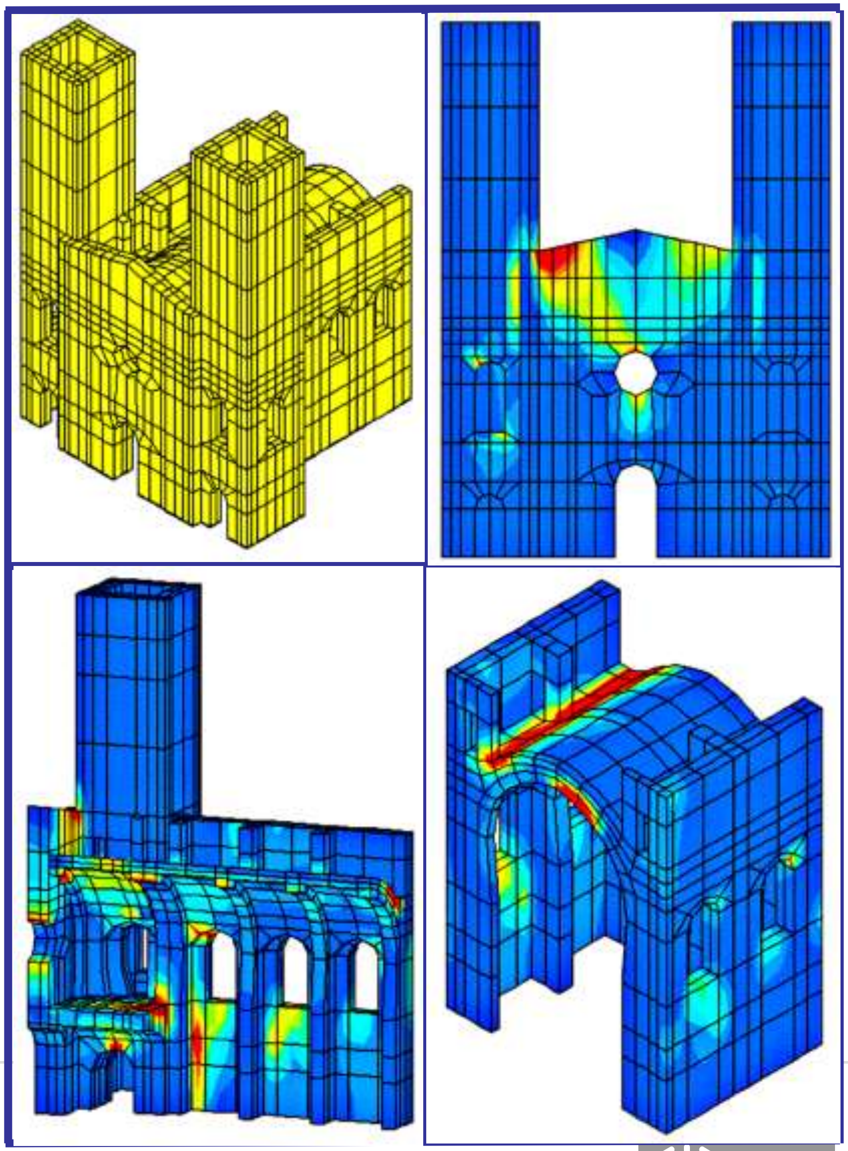
Holy Christ Church, Outeiro



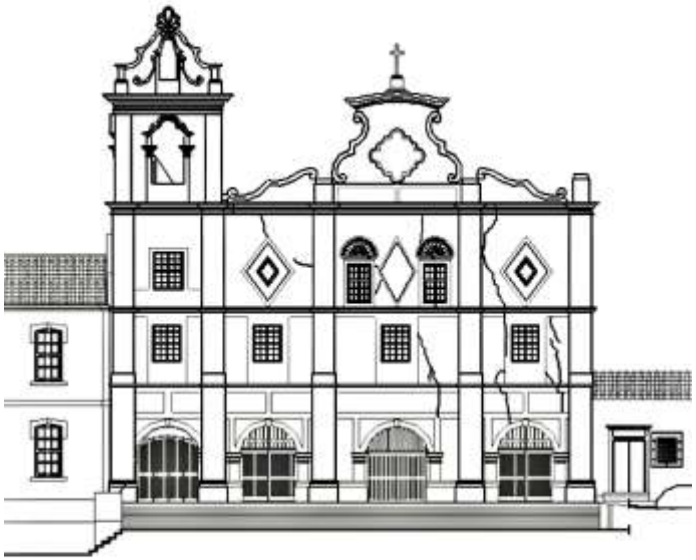


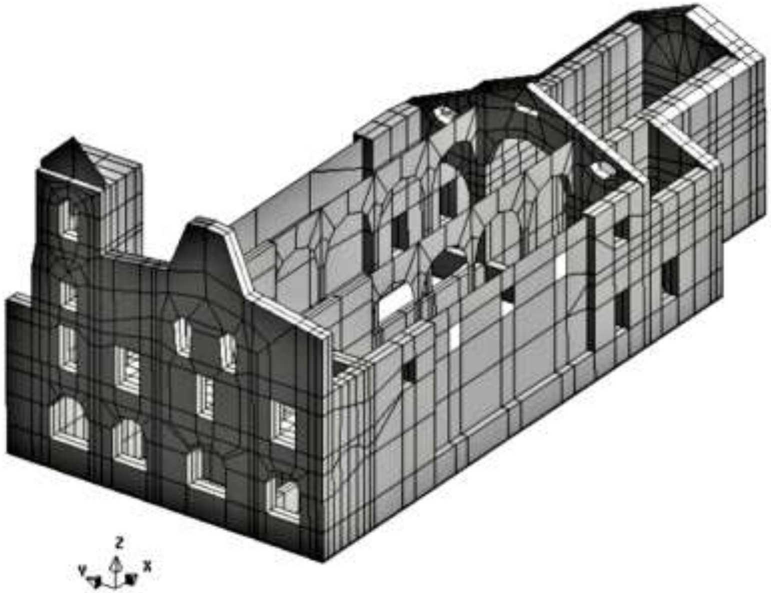


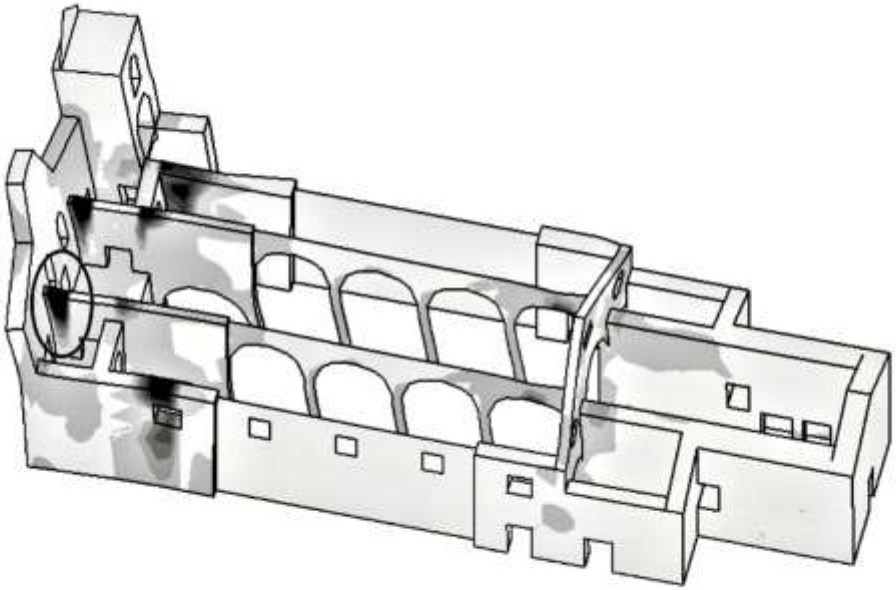
São Torcato Sanctuary, São Torcato



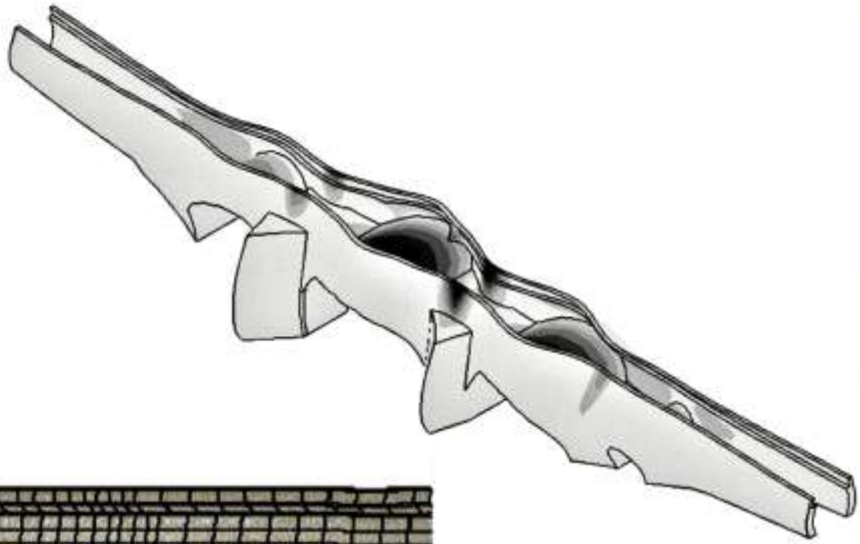
Saint Francis Church, Horta, Azores



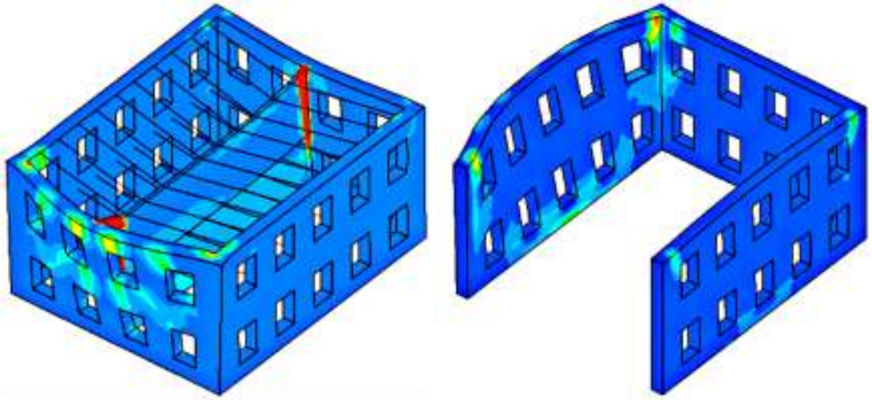
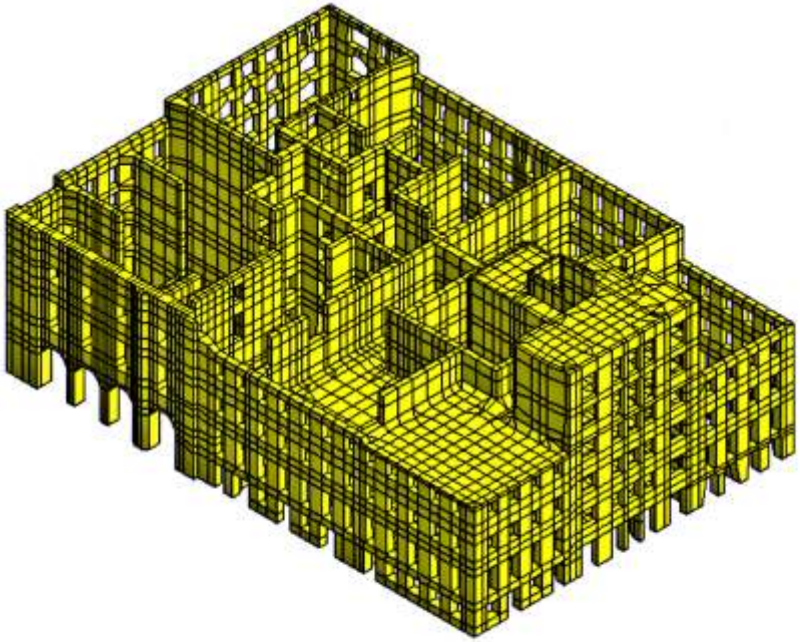
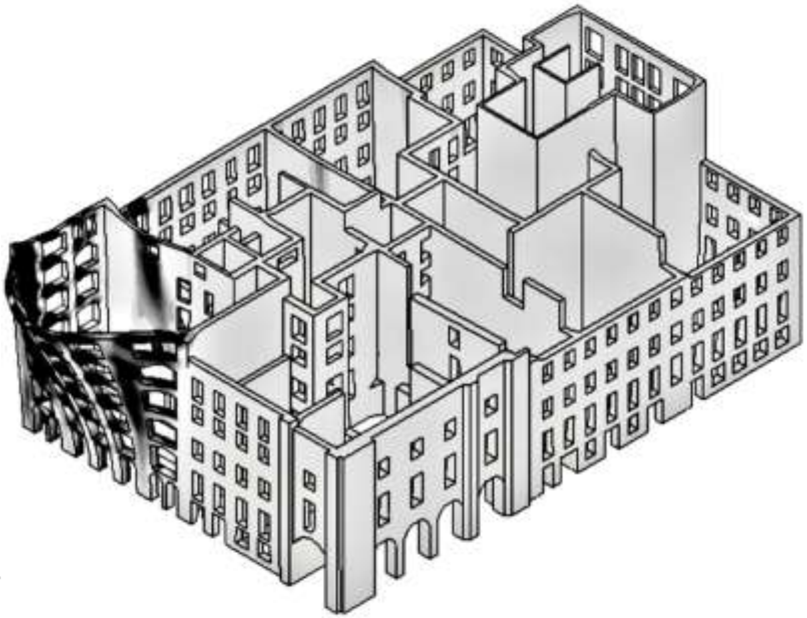




Donim Bridge, Donim



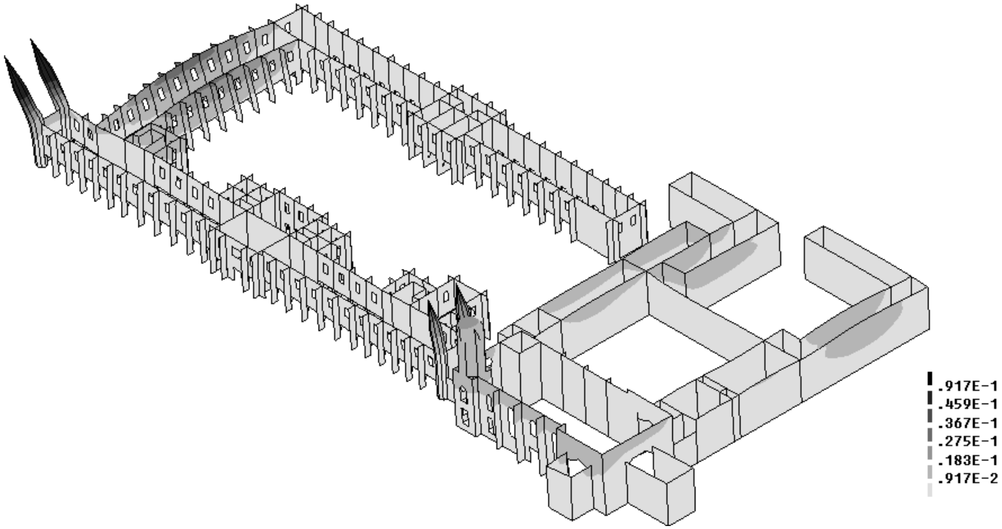
Typical downtown construction of Lisbon

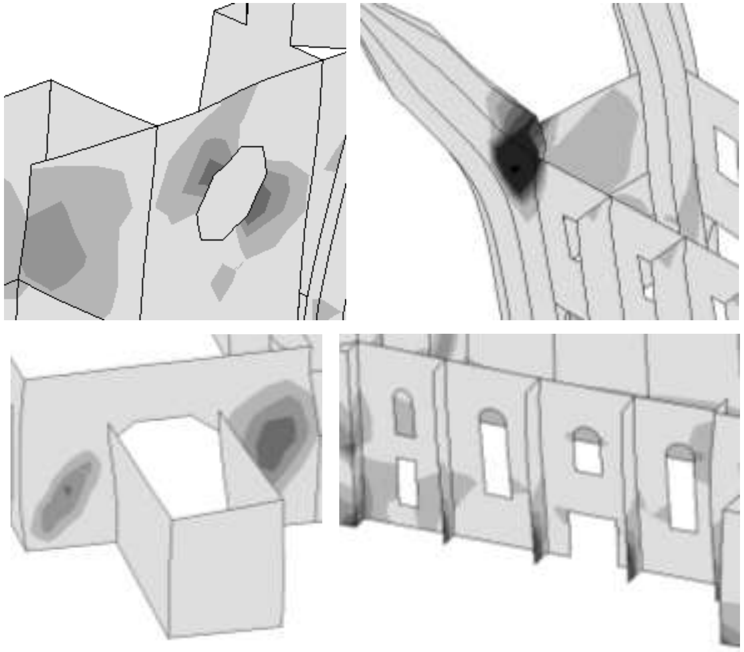
Monastery of Jerónimos, Lisbon

Full model with 135.000 dof

Pushover analysis

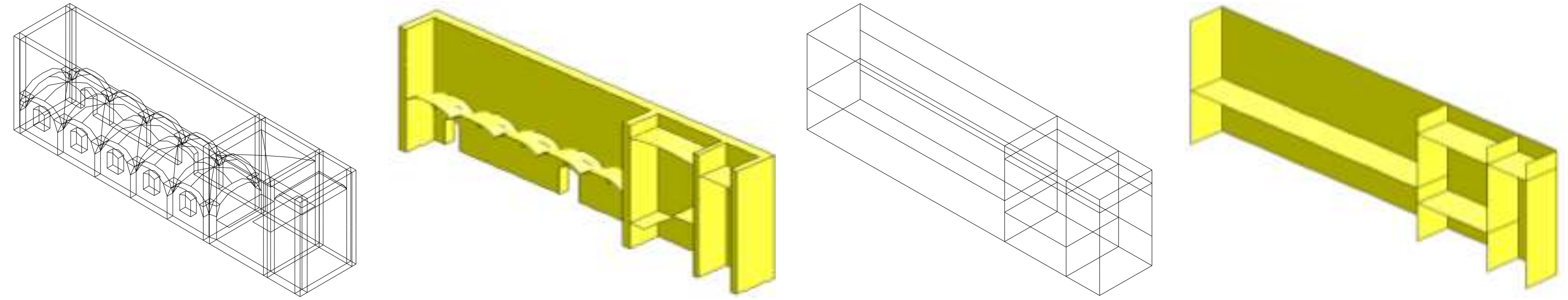


Deformed mesh

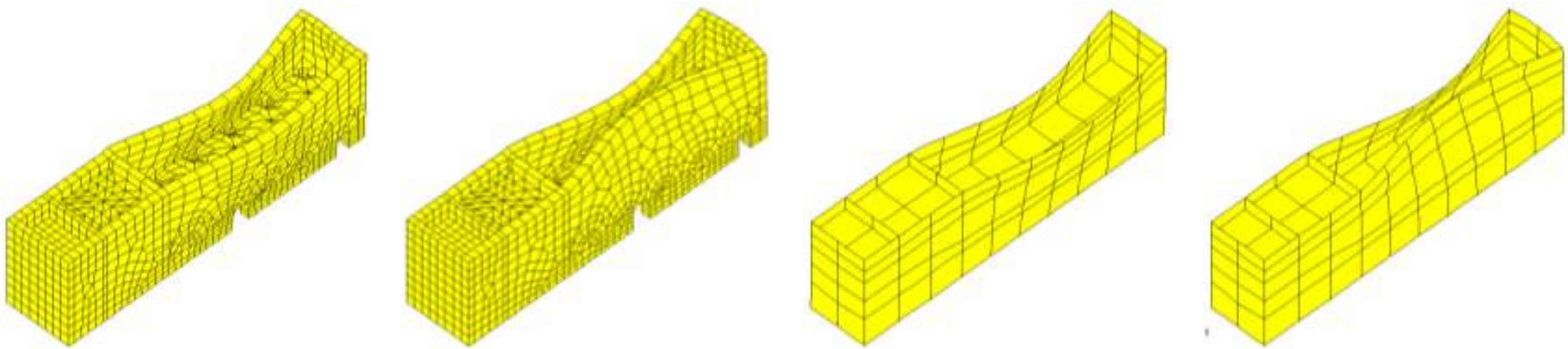


Details

Modeling 2.5D vs 3D



Evaluation of the modeling strategy

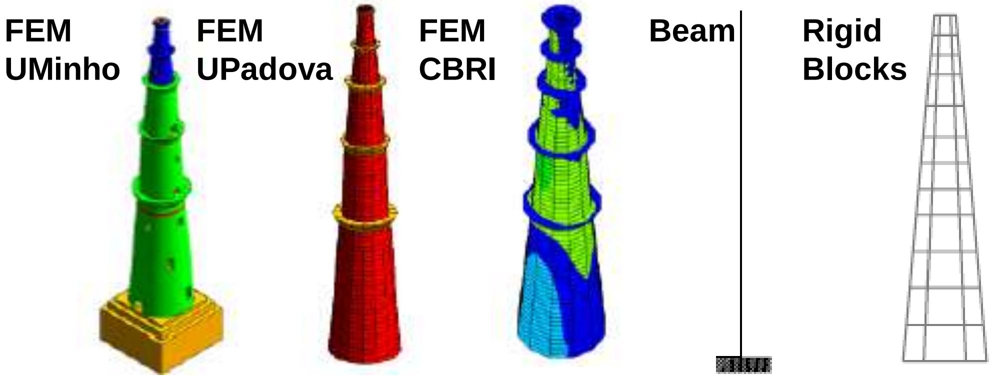


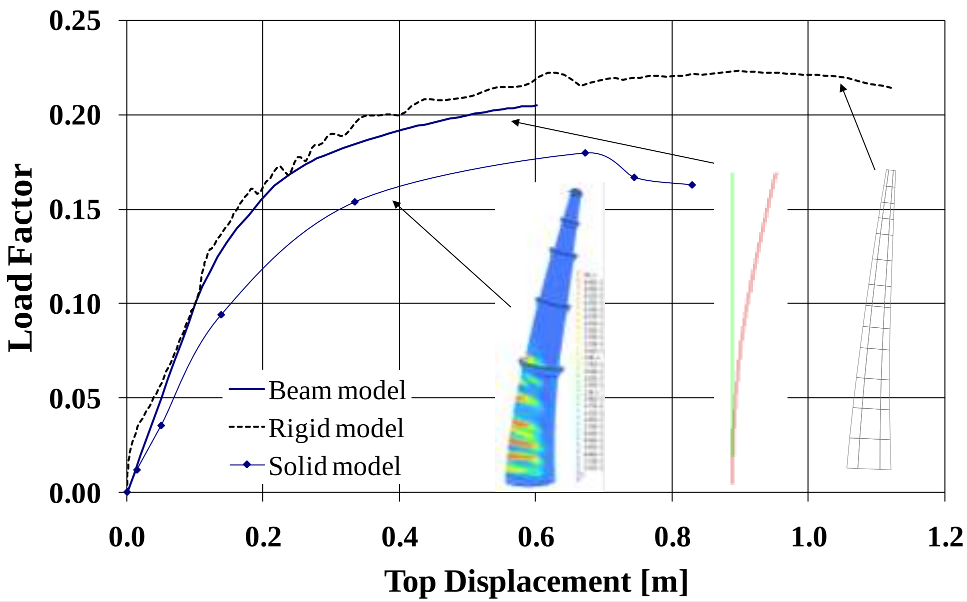
Comparison of first two modal shapes

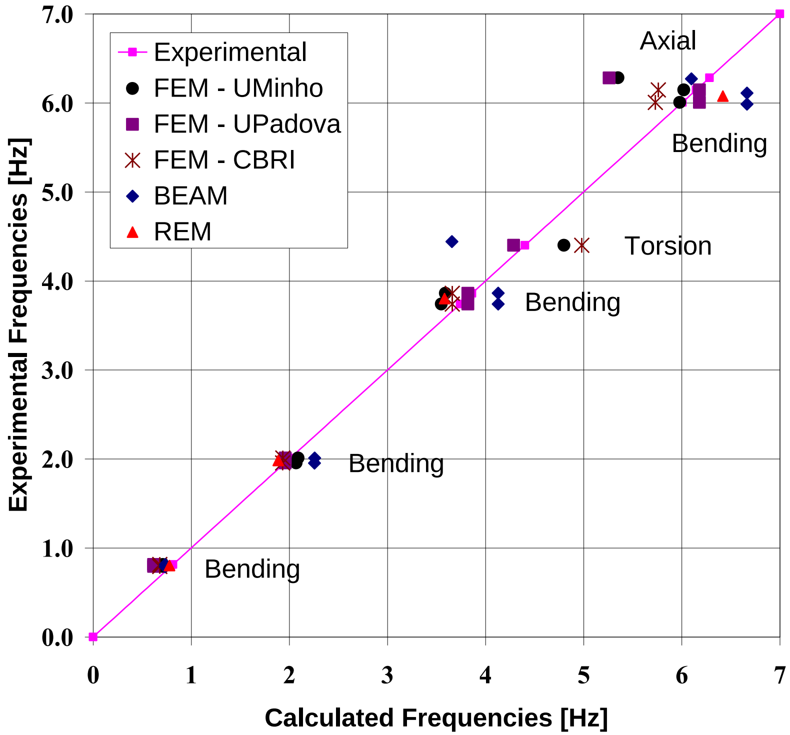
Refined model	$f_1 = 1.79$	$f_2 = 2.26$	$f_4 = 3.34$	$f_5 = 3.78$	$f_7 = 4.70$	$f_8 = 5.41$
Simplified model	$f_1 = 1.61$	$f_2 = 2.41$	$f_4 = 3.25$	$f_7 = 3.98$	$f_9 = 4.39$	$f_{12} = 5.31$

Comparison of first six global modal frequencies

Material vs. Model Properties







Calibration

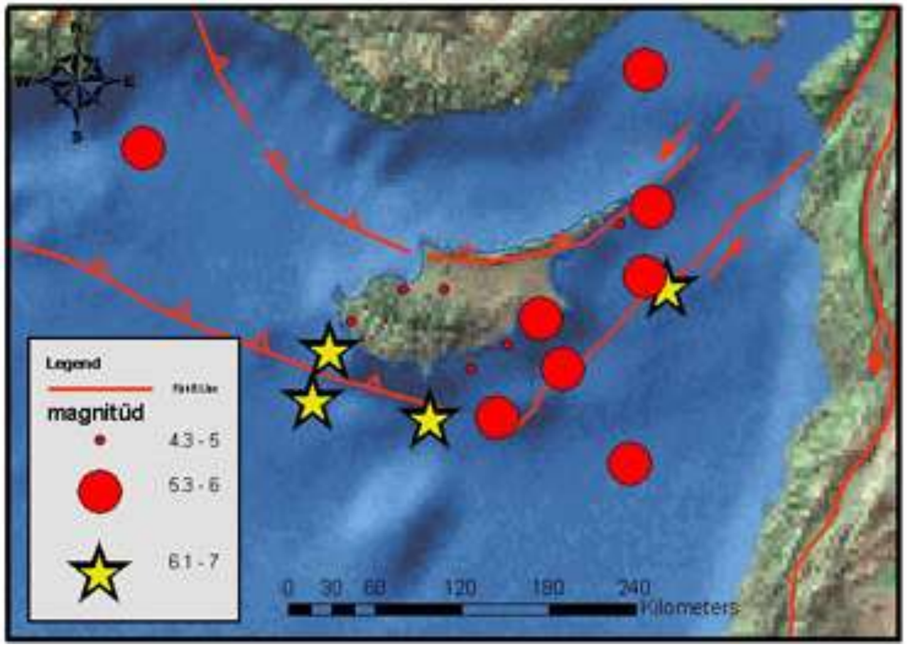
Elastic parameters and capacity depend on the different models

Application #1: Famagusta, Cyprus

- Complex political situation
- Advanced deterioration and abandonment, with inclusion in the 100 Most Endangered Sites (WMF, 2008)
- The old city has many churches and an impressive wall

Local seismicity

- Complex tectonic area, in the intersection of three plates: African, Eurasian and Arabic
- Alpine-Himalaya region is the 2nd most active in the world, with 15% of the world activity
- Cyprus had 16 large earthquakes in the last 2000 years (intensity VIII or more)
- The city was destroyed in the 1st and 4th centuries. Much damage in 1924 e 1941 (earthquakes with Mw > 6.0)



Strong earthquakes between1896 and 2000

(a)

(b)

Current condition: (a) garbage and misuse; (b) stone deterioration

(c)

(d)

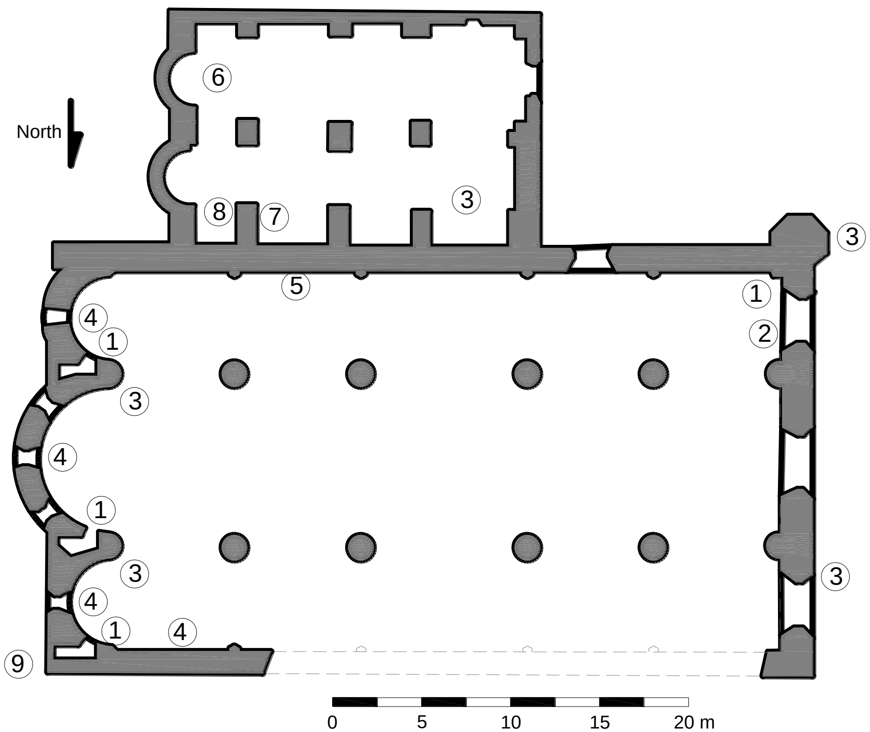
Current condition: (a) loose stone elements; (b) corrosion in r/c lintel

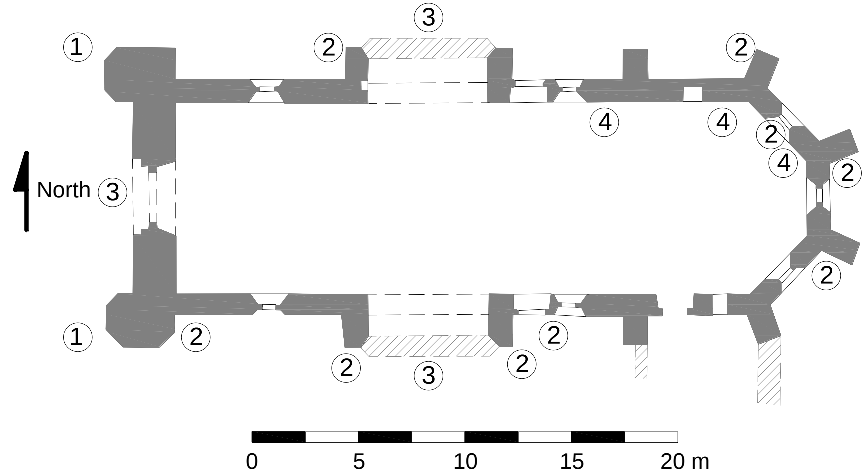
(e)

(f)

Current condition: (a) deficient buttresses; (b) tower rotation

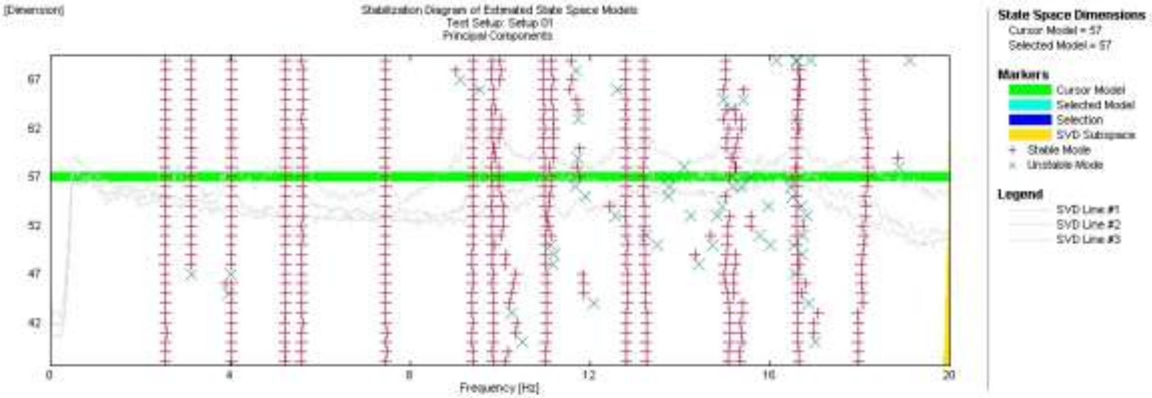
Conservation project for 3 churches



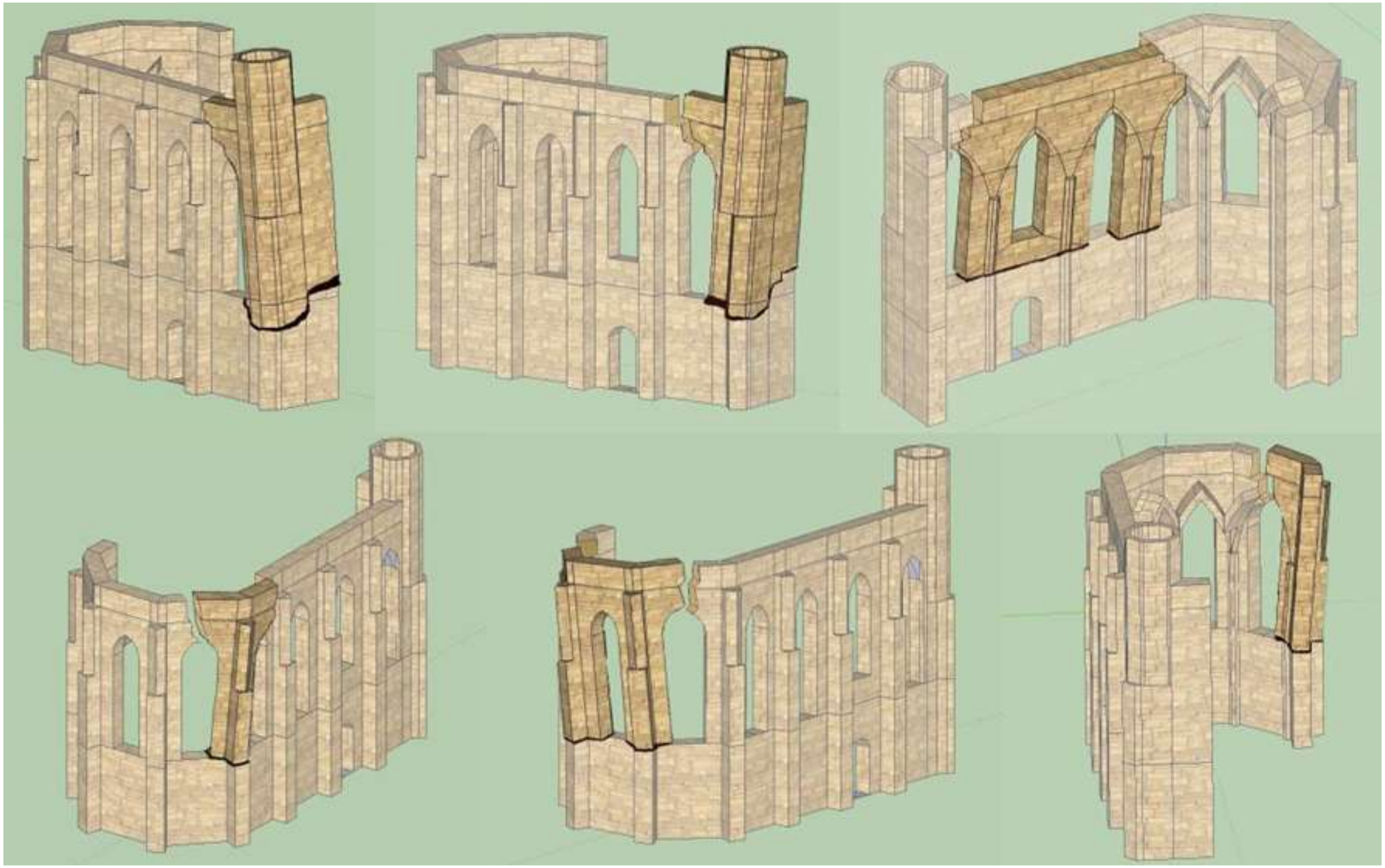


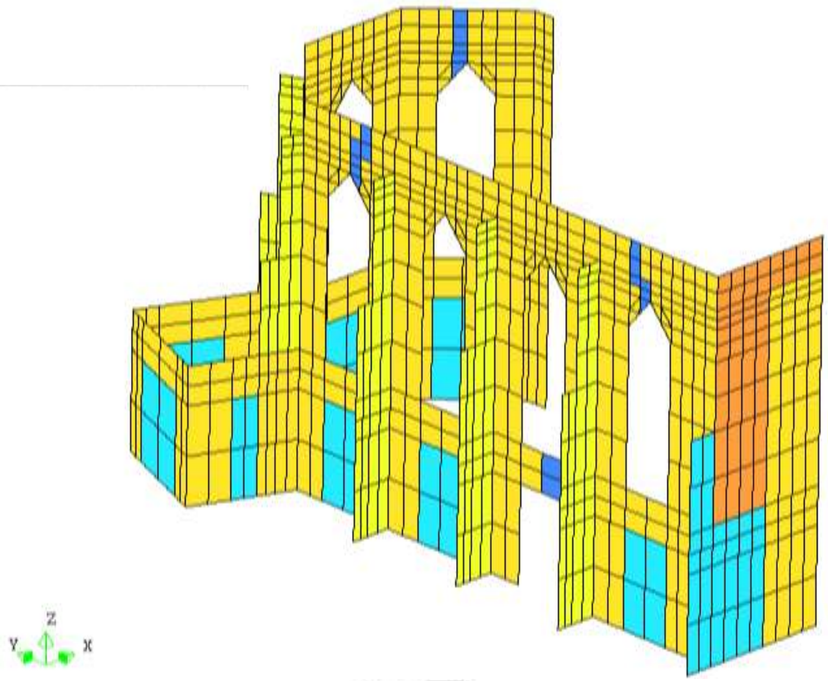
(1) Gate to prevent access; (2) joint repointing; (3) conservation; (4) frescoes and engravings protection; (5) consolidation of window tracery; (6) vault consolidation; (7) arch consolidation; (8) cleaning; (9) flying arch consolidation

In situ testing



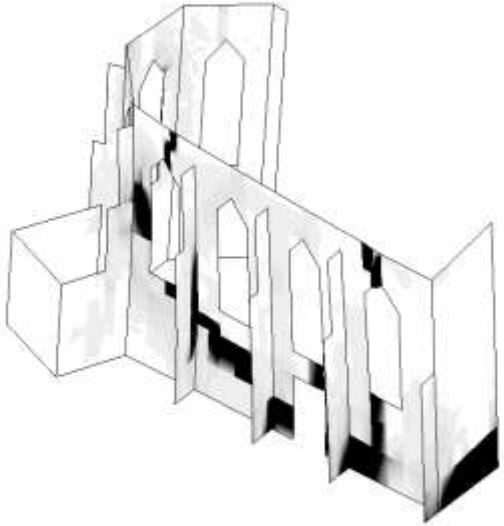
Seismic safety assessment



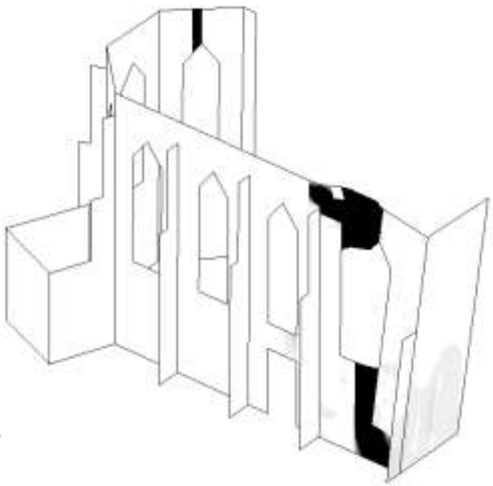


Mode Shape	f_{EXP} [Hz]	f_{NUM} [Hz]	Error [%]	MAC [%]
1st	2.57	2.43	-5.45	94.2
2nd	3.14	3.19	1.59	49.8
3rd	3.95	3.95	0.00	40.8
4th	5.26	5.29	0.57	76.5

FE Model, frequencies and mode shapes

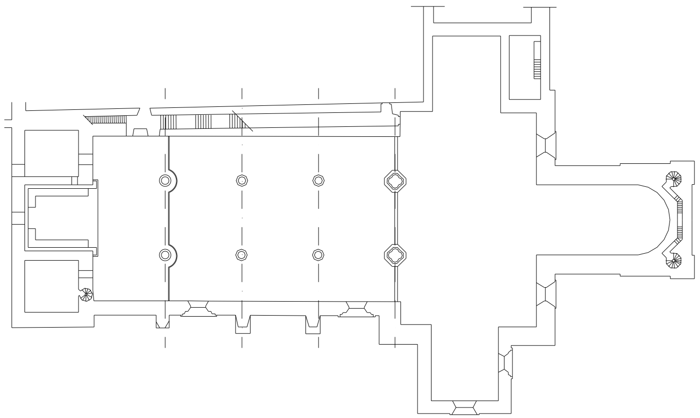


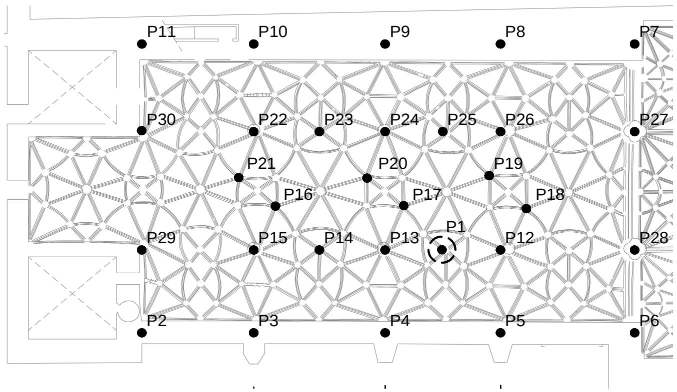
+x



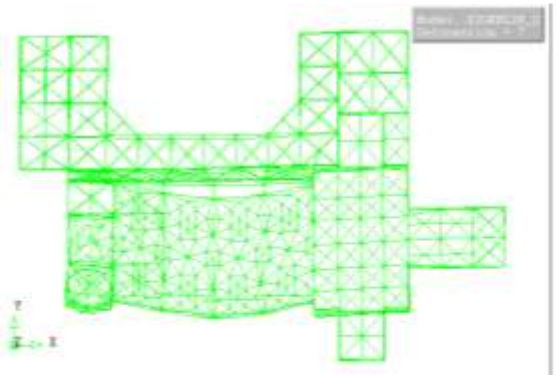
-y

Application #2: Monastery of Jerónimos, Lisbon

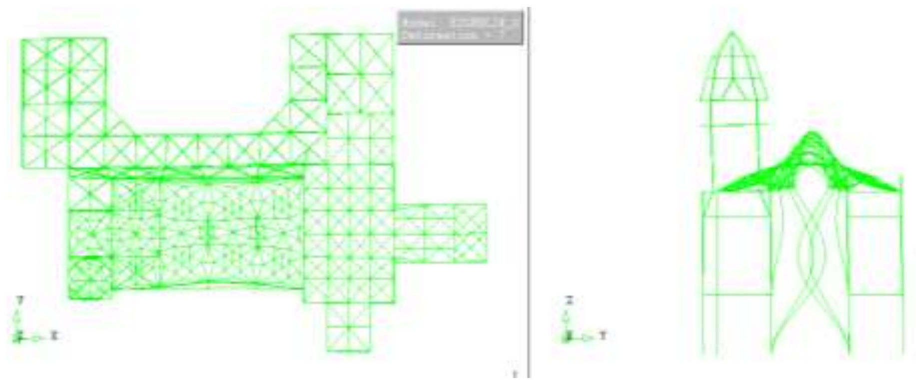




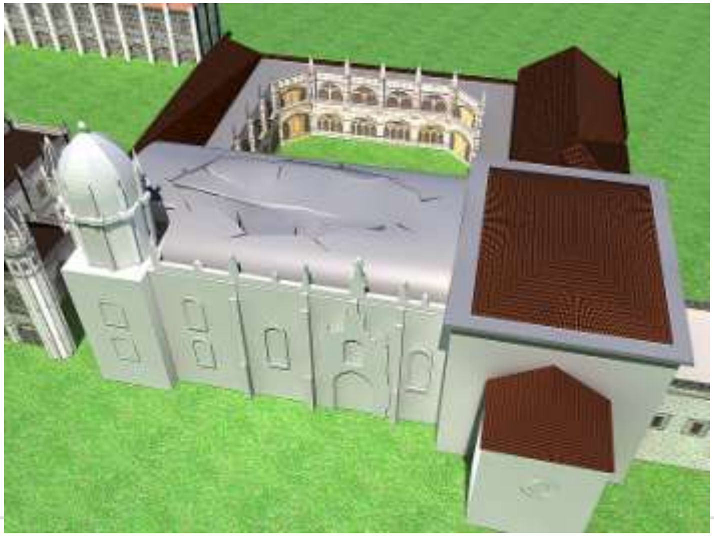
Nave and columns identified separately (columns much larger E)



Mode $1_{num}/1_{exp}$ (3.8/3.7 Hz)



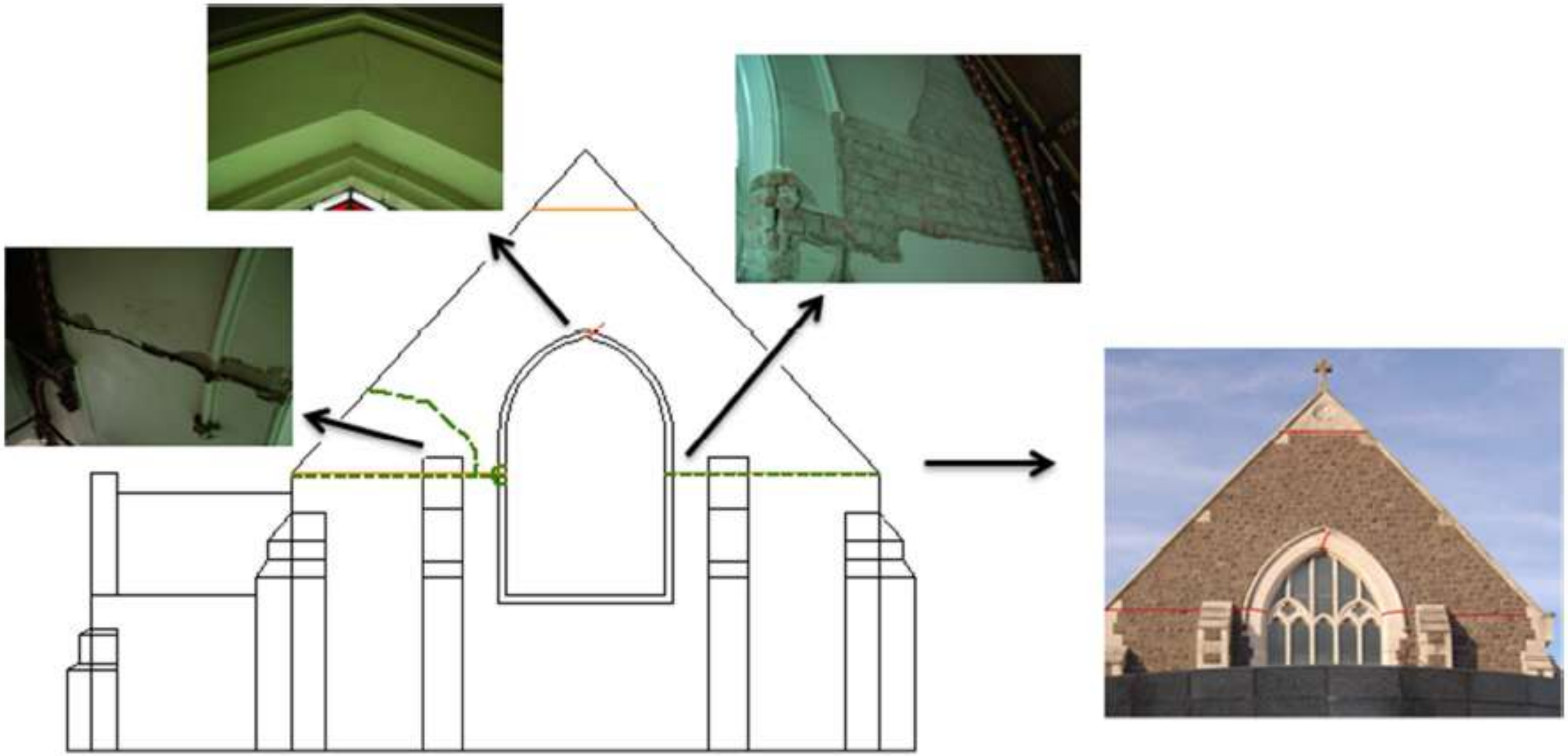
Mode $4_{num}/2_{exp}$ (5.3/5.1 Hz)



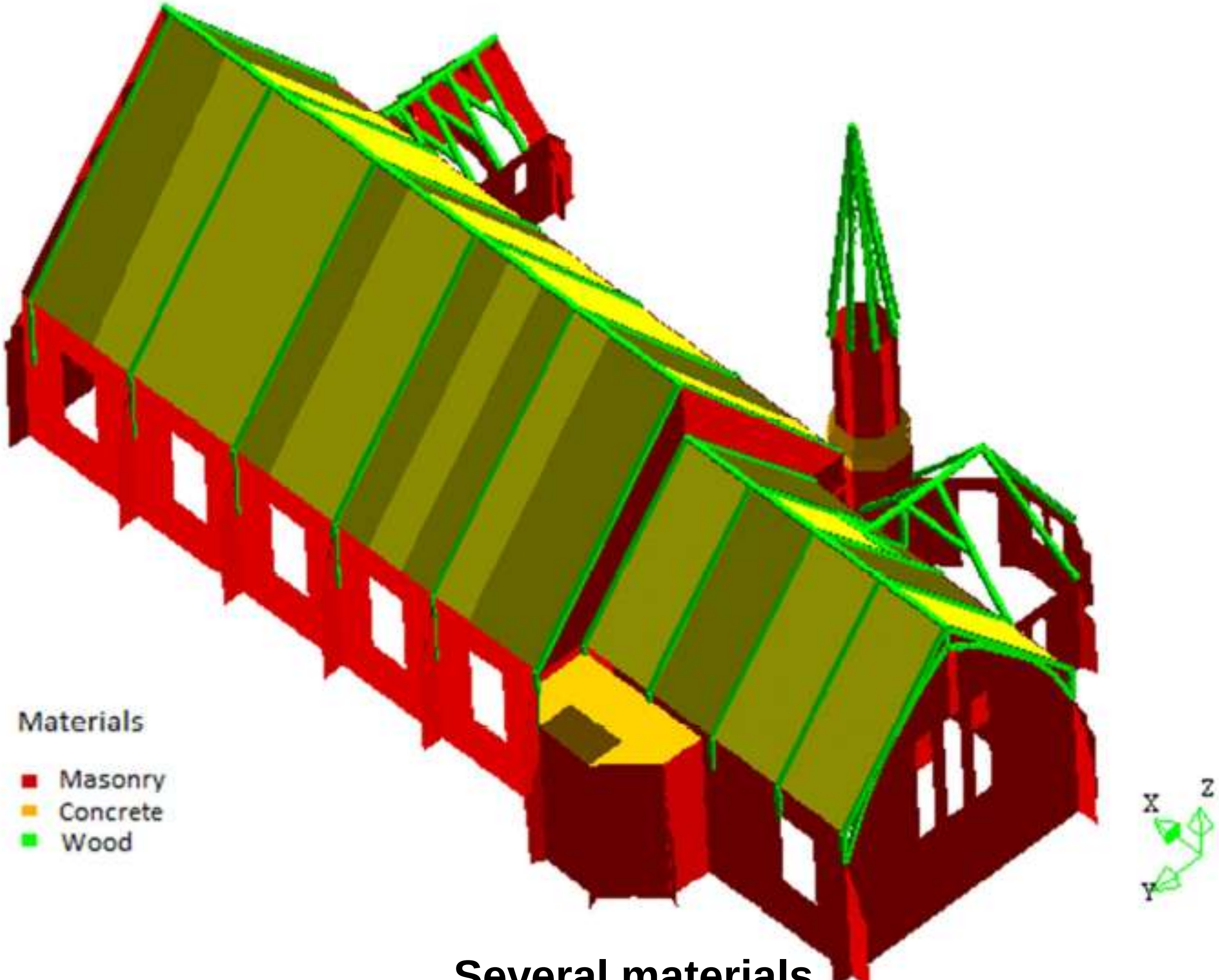
Collapses / Safety Assessment

Application #3: St. James Church, Christchurch, NZ

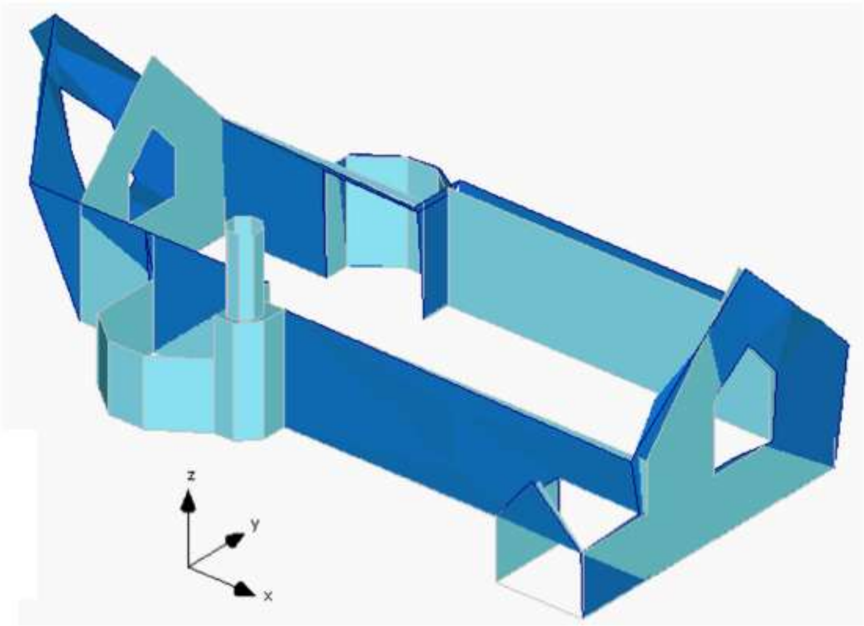
Structure Damaged in the February 2011 Earthquake

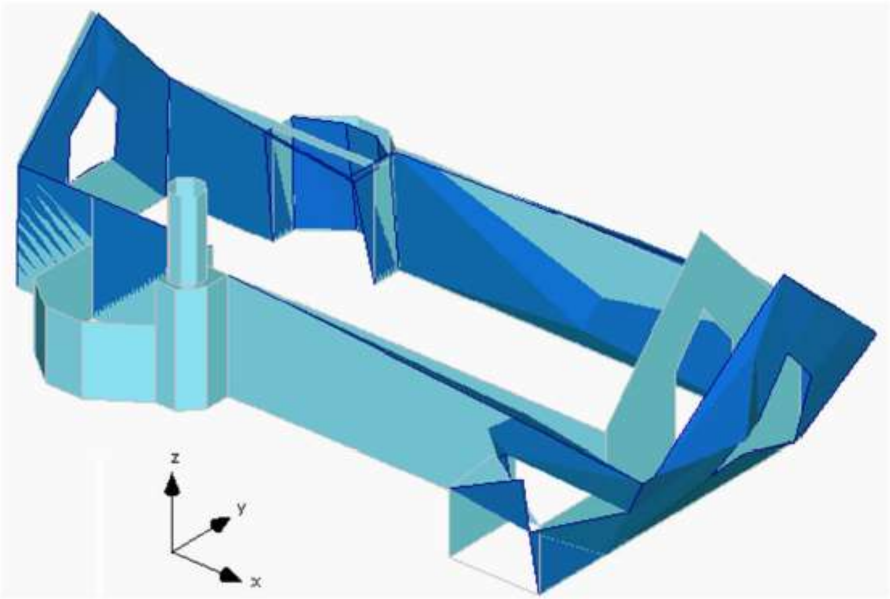


Several cracks



Several materials





Mode shapes of isolated elements (1st and 2nd mode)

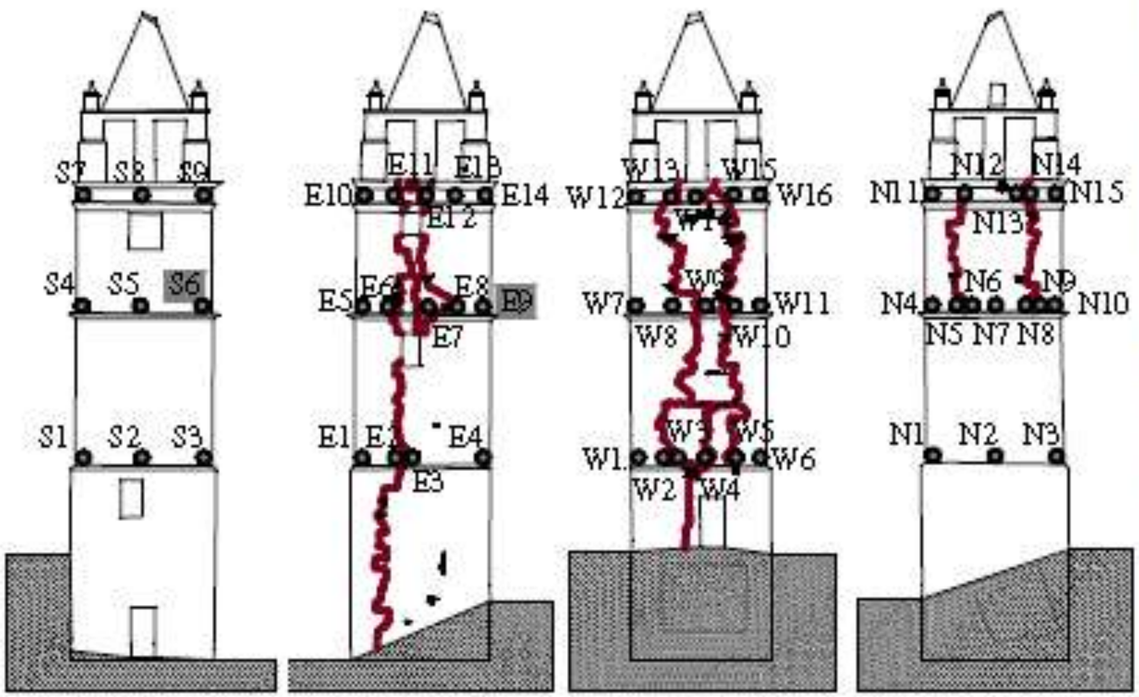
Case Study #4: Mogadouro Tower

- Built after 1559. Rectangular section 4.7 × 4.5 m^2 and height of 20.4 m. Large granite stones in the corners and rubble stone with thick lime mortar joints in the central part of the walls. Thickness of walls is about 1.0 m.
- In 2004, the tower was severely damaged, with large cracks, material deterioration and loss of material in some parts. Conservation works carried out in 2005 reinstated the tower safety, including lime grout injection for the walls, filling of voids and losses, and installation of steel belt at two levels.

Before

After

Case Study #4: Mogadouro Tower



C

Dynamic tests and example of sensor locations

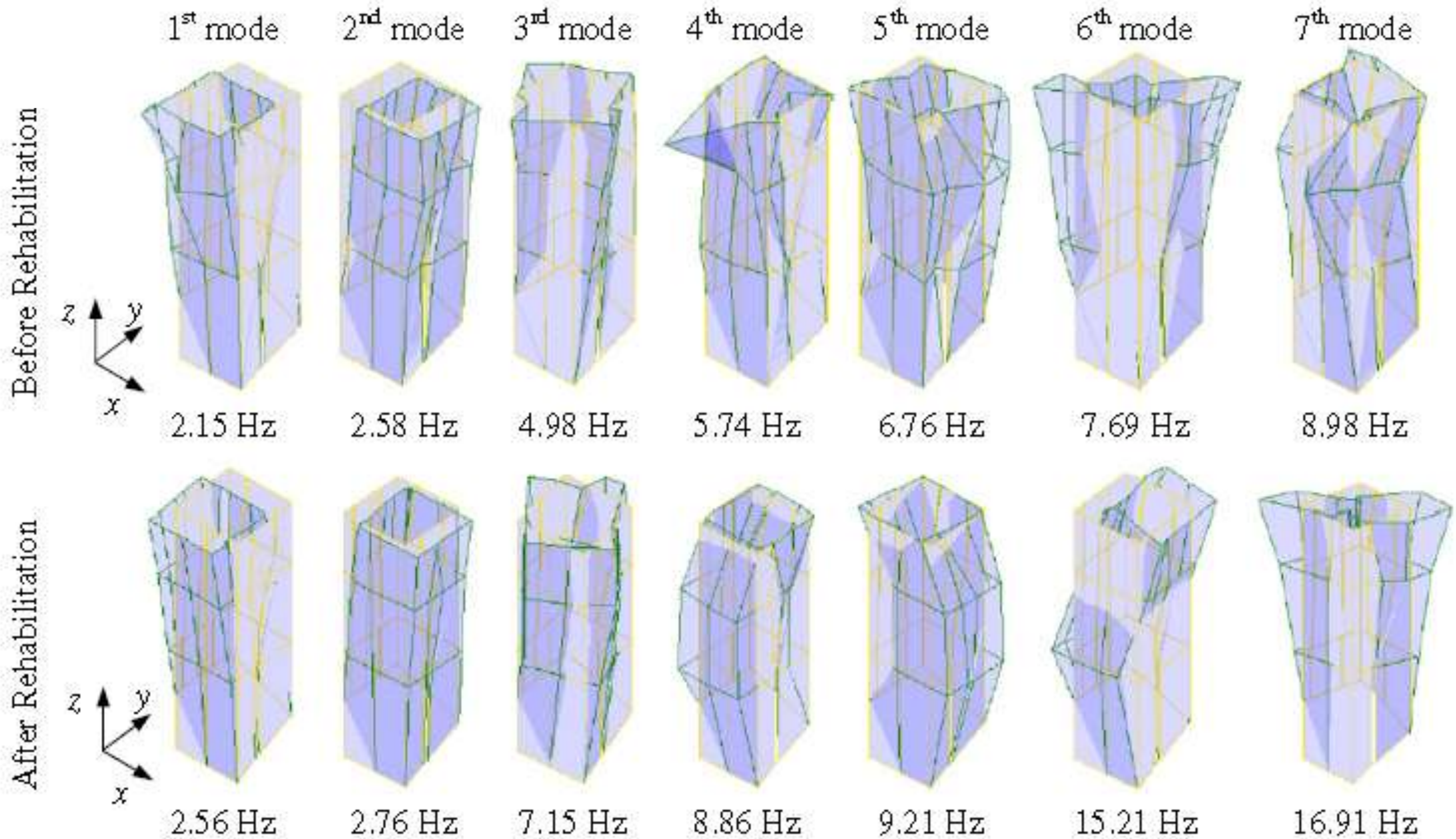
Case Study #4: Mogadouro Tower

Mode shape	Before		After		Δ_ω	Before		After		Δ_ξ
	ω (Hz)	CV_ω (%)	ω (Hz)	CV_ω (%)	(%)	ξ (%)	CV_ξ (%)	ξ (%)	CV_ξ (%)	(%)
1st	2.15	1.85	2.56	0.21	+19.28	2.68	219.51	1.25	0.13	53.26
2nd	2.58	1.05	2.76	0.30	+6.70	1.71	94.02	1.35	0.17	21.00
3rd	4.98	0.69	7.15	0.27	+43.67	2.05	65.33	1.20	0.14	41.32
4th	5.74	1.56	8.86	0.47	+54.37	2.40	24.27	1.31	0.13	45.72
5th	6.76	1.13	9.21	0.21	+36.13	2.14	31.74	1.16	0.12	45.65
6th	7.69	2.94	15.21	2.24	+97.87	2.33	55.98	2.54	0.24	+9.11
7th	8.98	1.21	16.91	1.40	+88.27	2.30	46.39	1.49	0.23	35.07
Average values	–	1.49	–	0.73	+49.47	2.23	76.75	1.47	0.17	40.34*

* Average value calculated only with negative differences.

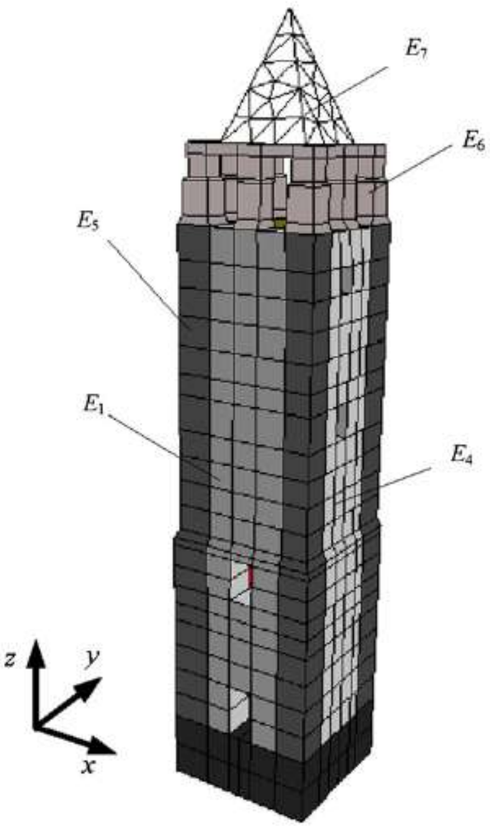
Dynamic response before and after the conservation works

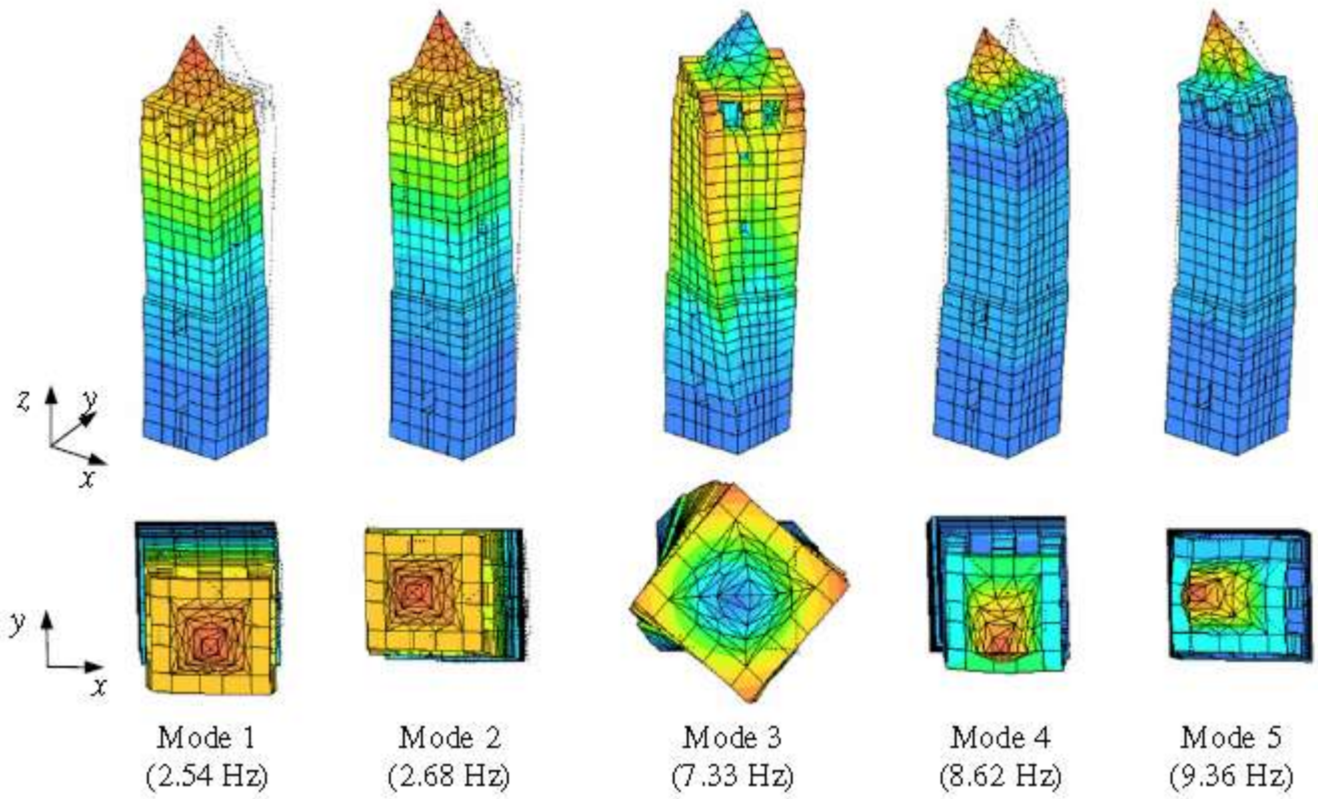
Case Study #4: Mogadouro Tower



Experimental mode shapes

Case Study #4: Mogadouro Tower

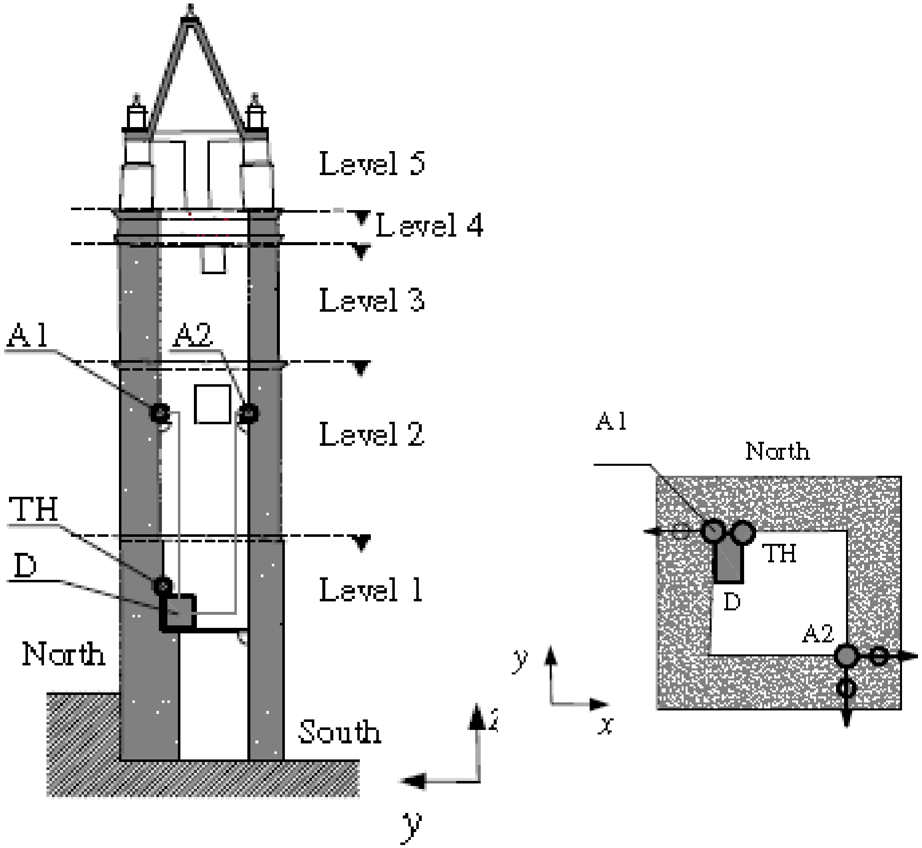




Updating parameters
(shell model with bad results. Foundation was needed)

Numerical results
(average error in frequency of 2% and average MAC of 0.98)

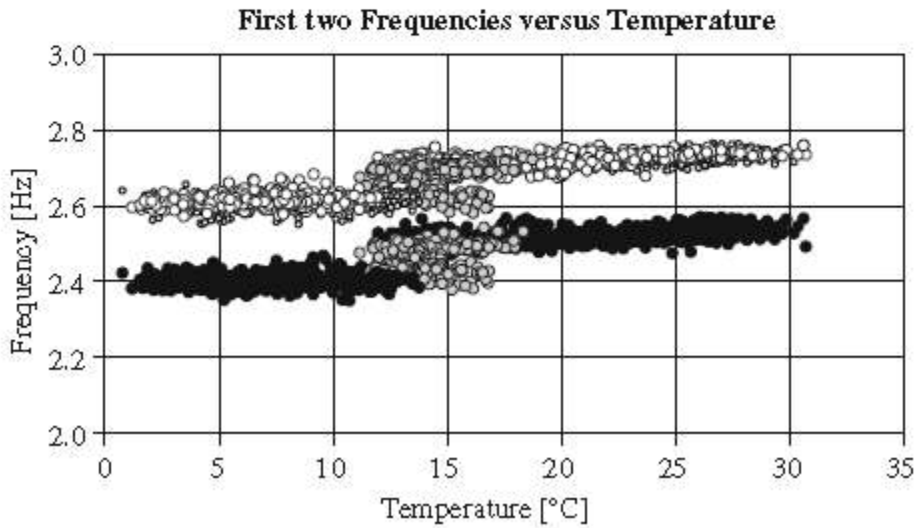
Case Study #4: Mogadouro Tower

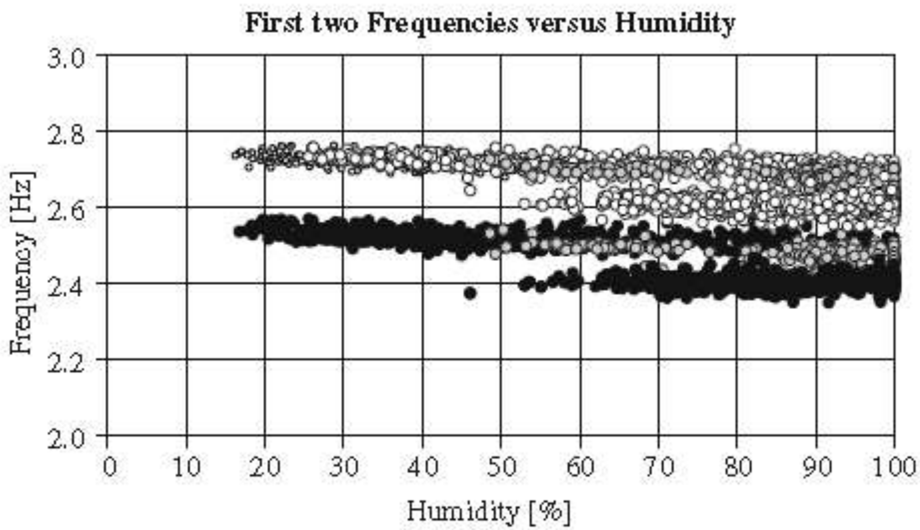


Dynamic monitoring

- ❑ **Measurements between April 2006 and December 2007**
- ❑ **No triggering (regular intervals)**
- ❑ **Automatic modal estimation based on SSI**
- ❑ **Procedures to avoid unrealistic modes**

Case Study #4: Mogadouro Tower





Results
(4% change in frequency in the beginning of the raining season)

Conclusions

isise

Universidade do Minho

Conclusions

- Conservation Engineering is a complex and exciting field. Specific tools and knowledge for the discipline are available

- Time shows that many historical masonry constructions collapsed due to extreme events (e.g. earthquakes). Fatigue and strength degradation, accumulated damage due to traffic, wind and temperature loads, soil settlements and the lack of structural understanding of the original builders are high risk factors for cultural heritage buildings.

- Structural analysis and safety assessment of historical masonry buildings are necessary. Structural modeling play a key role here.

Local and global models for seismic safety assessment

Paulo B. Lourenço

pbl@civil.uminho.pt
www.civil.uminho.pt/masonry

Universidade do Minho